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THE HARBOR WORK AT MANILA, Philippine Islands, is to be let on Aug. 1, 1901. This contract will include the completion of the east and west breakwaters—commenced under Spanish rule; the building of a pile bulkhead, 4,700 ft. long, with piles furnished by the government; and dredging a basin and a 30-ft. channel to deep water. The approximate quantities are: Dredging, 5,000,000 cu. yds., of sand, shells and mud; riprap, 150,000 cu. yds.; concrete and rubble masonry, 21,000 cu. yds., the stone to come from government quarries at Mariveles or Biumgonau, about 35 miles from the city of Manila. A crushing plant, steam launches and lighters will have to be provided by the contractor. The required bond is $100,000 in gold. The conditions are so complicated that it is to be regretted that the time is hardly sufficiently long to permit the sending of an engineer to the spot to first study the problem; as this precaution would be a necessary preliminary to an intelligent bid. Labor in Manila will cost at least $1.00 (Mexican) per day; though coolies could be brought from China for half that sum, and are better workmen. Transportation is costly: a 30 to 40-ton lighter will cost about $25 (Mexican) per day; and will consume two days in bringing one load of rock to the work. Steam launches cost from $35 to $60 (Mexican) per day, and will tow two or three loaded lighters; but these launches and lighters are both exceedingly hard to get.

RAISING THE "MAINE" from the bed of the harbor of Havana, Cuba, where it has been lying since it was blown up in 1898, is to be attempted by N. F. Chamberlain, of Chicago, who has a contract for this work from the United States officials at Havana. The wreck has sunk deep in the soft mud, but the proposal is to build a watertight cofferdam enclosing the wreck, and to pump out the mud and water to enable the hull to be patched and pontoons attached to the vessel. Water would then be admitted, and the wreck raised by the buoyancy of the pontoons. The cofferdam, according to reports is to consist of two rows of 70-ft. piles, the piles of each row being connected by a woven mattress work to retain a filling of sand and stone between the two rows. The naval officials would not allow the wreck to be blown up by dynamite, partly out of respect for the bodies of those who went down with the ship, and partly for fear of exploding the ship's magazines, with disastrous results to the buildings along the water front.

SABLE ISLANDS, about 90 miles off the coast of Nova Scotia, is fast being washed away, and the Canadian Department of Marine and Fisheries will attempt to arrest erosion by tree planting. The island is about 21 miles long and is in the form of a slender open crescent, one mile wide at the widest part. Within the memory of present residents the sea has washed away several miles on the west end, and the lighthouse has been moved twice in 15 years. Early surveys show that the island was once 40 miles long. As dangerous shoals extend on all sides, with strong ocean currents setting upon them, and there is a prevailing fog about them, it is essential that light and life and signal stations be maintained. To this end over 80,000 evergreens are now being planted, with the hope that they will stop further decrease of land area. Between 1583 and February, 1899, no less than 170 known wrecks are recorded for the island. At present there are six life-saving stations and two lighthouses on the island, employing 18 men; and these, with their families, make up a total population of 45 souls, who occupy wooden buildings erected by the government.

A STEEL AND CONCRETE TUNNEL intended to be used to supply condensing water to the 39th St. powerhouse of the New York Edison Co. was recently launched at the Crescent shipyards, at Elizabethport, N. J. The tunnel was designed by Haight & Lande, who are also the inventors and patentees. It is 225 ft. long, 12½ ft. in diameter and weighs 1,200 tons. It consists of exterior and interior shells of ¼-in. steel, between which is 15 ins. of concrete. The tunnel will be towed to the East River, where it will rest on a foundation of piles and rip-rap 23 ft. below the surface. The cost was $75,000.

THE "VIRGINIA" TYPE OF BATTLESHIP has been recommended by a board of naval experts, consisting of Rear-Admiral R. D. Evans, Naval Constructor D. W. Taylor, Capt. C. F. Goodrich, Commander Richard Wainwright, Prof. P. R. Alger, U. S. N., Lieut.-Commander J. B. Murdock, and Lieut. S. E. W. Kittelle. A majority of the board has reported against the construction of armored cruisers of the "Maryland" type. They agree that more vessels of the "Virginia"' type are needed by the United States navy. The "Maryland" is a sister ship to the "Colorado" and "South Dakota," all three of which have been appropriated for by Congress. They are of 13,680 tons displacement, 502 ft. in length, and have two barbette turrets and two military masts. They will have a speed of 22 knots and will carry four 8-in. guns, fourteen 6-in. rapid fire guns, two 3-in. rifles, eight one-pounders, and two submerged torpedo tubes. The battleship "Virginia" is a sister ship to the "Rhode Island" and is to have a displacement of 14,948 tons and a speed of 19 knots. She will carry four 12-in. rifles, eight 8-in. rifles and twelve 6-in. rifles in the main battery. Rear-Admiral Evans advocates a type of battleship of 16 knots, with 15,000 tons displacement and carrying four 13-in. guns and twelve 8-in. guns, without turrets.

THE 100-MILE LAKE STEAMER RACE, between Cleveland and Erie, ended in a victory for the Cleveland passenger steamer "Erie," which made the distance in 4 h. 19 m. 9 s., beating the Detroit passenger steamer "Tashmoo" by 45 seconds. The speed of the boats approximated 23 miles per hour; and both boats were especially fitted out and prepared for the race, with a limited number of spectators on each, and no deck freight.

THE DREDGE SAMSON, built for the Queensland government on the designs of Mr. Lindon Bates, was officially tested on April 26 in the Brisbane River. In this trial the dredged material was discharged through 1,500 ft. of floating pipe and the cut made was 30 ft. wide and 6 ft. deep. The report of the Marine Department shows an advance in 53 minutes of 1,019 lin. ft. and 8,150 cu. yds. of material excavated. This was 3,000 cu. yds. in excess of the guarantee. The dredge was built in England, but the engines and winches for operating the dredge were supplied by the Lidgerwood Manufacturing Co., of New York.

THE LONDON UNDERGROUND RAILWAY is to be equipped with electric power by a syndicate organized by Mr. Charles T. Yerkes, of Chicago, and called the Metropolitan District Electric Traction Co. The necessary capital will be supplied by American and English financiers. The Maryland Trust Co. will be the American financial agents of the enterprise. A generating station will be erected at Chelsea, and the necessary changes will be made in the permanent way and the rolling stock. The latter will be of American pattern, using some system of multiple unit train control. It is estimated that the work will be finished in two years.

THE PENNSYLVANIA RAPID TRANSIT CO. is the name selected by the promoters of the Focht and Emery transit bills passed under pressure by the Pennsylvania Legislature on June 4. This company proposes to build a double-track surface line, with underground or overhead trolley, the whole length of Broad St., Philadelphia; also an elevated railway from Front St., on the Delaware, out Market St. to the county line, with branches out Ridge Ave. to Manyunk, and out Lancaster Ave. and out the Derby Road, etc. In opposition to this bill the friends of the Union Traction Co., now controlling the street railways of Philadelphia, are preparing to file articles of incorporation authorizing them to lay tracks on all streets 40 ft. or more wide, not now occupied by tracks. No estimates of cost are given, nor is it stated where the money is to come from for the new elevated and surface lines proposed by the Pennsylvania Rapid Transit Co.

THE CONCRETE RETAINING WALLS for the track-elevation work of the Chicago & Western Indiana R. R. are of approximately triangular section, the base width being half the height, while the top has a uniform width of 2 ft. The foundations are from 4 to 8 ft. deep. The face of the wall is vertical. This form of section is much more quickly and easily built, and requires less expense for the forms, than a wall having the back stepped in offsets. The method of constructing these walls was described and illustrated in our issue of Feb. 28, 1900.

THE FIRST RUSSIAN ELECTRIC SUBURBAN RAILway was opened for traffic lately between Lodz, in Russian Poland, and the neighboring towns of Zgierz and Pabianice. It is 13½ miles long, and uses Thomson-Houston motors. It was built at a cost of $560,000 by Polish merchants and manufacturers. The government receives a share of the profits, and the road and all its equipment become government property after 28 years.

THE NEW BOSTON ELEVATED RAILWAY was opened to traffic on June 9. This line extends from Dudley St., Roxbury to Sullivan Square, Charlestown, a distance of about 5 miles, and it cost $10,000,000. The first steel work was erected in November, 1899. The time made between terminals is about 20 minutes less than by the surface cars.

THE MOST SERIOUS RAILWAY ACCIDENT of the week occurred on the Delaware, Lackawanna & Western Ry. at Vestal, N. Y., on June 9. A double-header special freight train collided with the rear end of a regular freight train which was stopping for water. A quantity of dynamite in a car of the latter train exploded, completely wrecking both trains, killing five men and injuring seven.

THE RAILWAYS OF JAPAN, says U. S. Consul-General E. C. Bellows, of Yokohama, now aggregate 3,638 miles in length, including 2,713 miles of private lines. In addition to this the government has charters for 1,230 miles, and private individuals charters for 2,483 miles, not yet opened to traffic. The government has paid, for its 158 miles of double track road and 675 miles of single track, the sum of $34,700,166 gold, or an average of $41,670 per mile. The paid-up capital of the private lines amounts to $86,486,587 gold, or an average of $30,833 per mile. These railways are well patronized, and within the past year sleeping cars have been put on the line between Tokyo and Kobe; the charge for a berth being $2 gold, plus a first-class ticket. Dining cars are being built.

THE SWITCHING AND TRANSFER YARDS now being built at Chicago by the Chicago Transfer & Clearing Co. will include four separate parallel yards, each operated by gravity switching from a "camel-back" having grades of 0.9% in both directions from the summit. At the foot of each incline the tracks will diverge like a fan to the classification yards. Each "camel-back" will have five tracks, four for switching movements and one for through movements. The first of these four yards is under construction. The company owns a site of 4,000 acres south of 63d St., which will be occupied by the yards, warehouses, residences for the employees, etc. It is expected that several of the large grocery and commission houses, etc., in Chicago, will use these warehouses, so that they can replenish stock and fill orders without the trouble and expense of hauling all their goods by teams between the present city terminals and warehouses. The switching movements will all be controlled by the Westinghouse pneumatic-electric interlocking system of the Union Switch & Signal Co., the signals and switches being operated from a tower at the summit of the "camel-back." The yard now under construction is built on 2 ft. of sand filling, covered with a bed of slag, upon which will be the gravel ballast. The rails weigh 75 lbs. per yd., laid with four-bolt splice joints, and on oak ties. The frogs and switches are being made by the Cleveland Frog & Crossing Works. The water supply is obtained from driven wells 1,600 ft. deep, pumped by the air-lift system to a reservoir, and distributed to hydrants and engine water columns throughout the yards. For drainage a main sewer is being built, 18 ins. to 9 ft. diameter. Up to 30 ins. this sewer is of pipe, but for 36 ins. and upward it is of concrete. The concrete is mixed by machine on a work train, carried in wheelbarrows to the side of the trench and dumped into chutes. The centers remain in place about 48 hours, and the back filling is done by a drag scraper hauled by a wire cable on a machine traveling along one side of the trench, with a boom extending across to the ridge of excavated material on the other side. Two men guide the scraper as it takes its load, but it is dumped by a trip rope. The yards will be lighted by arc lamps on poles placed on each side of the "camel-back" and on the level, while a powerful searchlight on the signal tower will enable any particular spot to be lighted. The power house (for engines, pumps, air compressors and electric generators) is under construction. Mr. A. W. Swanitz is Chief Engineer.

CYLINDRICAL HEADLIGHT CASINGS conforming to the standard patterns of domes and sandboxes are being introduced on the Chicago, Milwaukee & St. Paul Ry. This form of casing is said to be more satisfactory than the ordinary rectangular casing, and is considered to improve the appearance of the engine by having a uniformity in the style of fittings.

A CONCRETE-FACED STONE DAM FOR THE ST. CROIX POWER CO. WISCONSIN.

About two years ago the St. Paul Gas Light Co., of St. Paul, Minn., decided to build a larger and modern plant and to develop one or more of the water powers within reasonable distance of that city. Upon investigation, the 30-ft. natural fall on the Apple River was selected, a fall 27 miles east of St. Paul, in the State of Wisconsin. The consulting engineers appointed were Mr. Henry Floy, of New York, and Prof. R. C. Carpenter, of Cornell University, practicing as Floy & Carpenter; and the following account of the interesting dam built is taken from a paper on the power plant lately presented by Mr. Floy to the American Institute of Electrical Engineers.

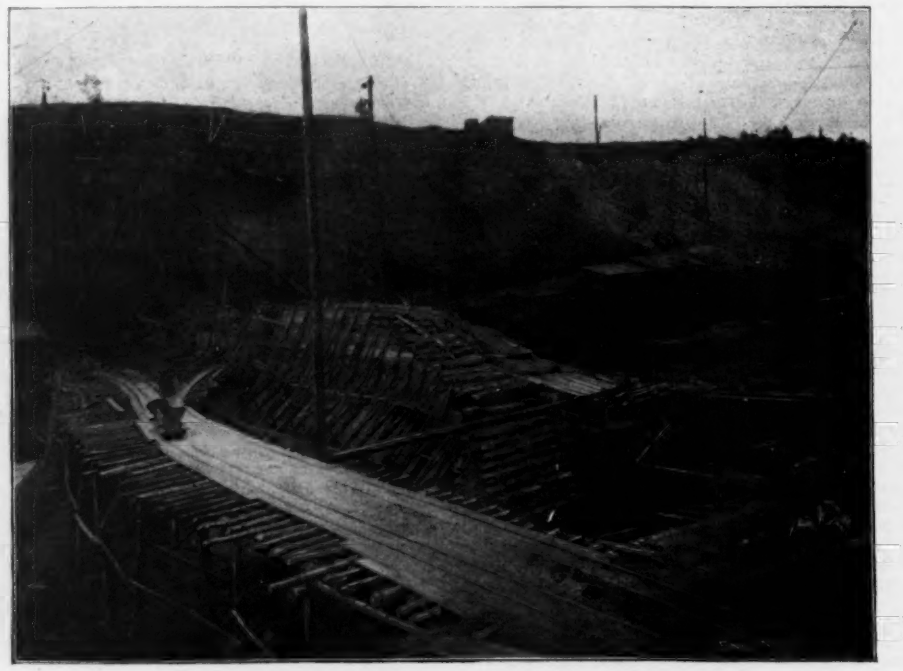


FIG. 3. VIEW SHOWING MASONRY CORE AND FORMS FOR CONCRETE FACING, ST. CROIX POWER CO.'S DAM.

It was found that above the fall a dam could be built to a height of 47 ft.; thus affording with a slight drop in the river below the fall a total head of 82 ft. With this head the minimum flow of the river would yield about 2,000 HP. continuously, and nearly twice that for the most of the year. To build the plant the St. Croix Power Co. was organized under the laws of Wisconsin, which contracted to sell all the current it could generate at this place to the St. Paul Gas Light Co.

The site of the dam was in a rough country, five miles from a railway, and the importation of building stone was out of the question. Consequently, the only material available was a brown sandstone, not hard enough to withstand the constant attrition of water. It was then decided to build the dam with a masonry core, resting on concrete foundations and enclosed in concrete facings. This masonry core was only carried within 6 ft. of the crest of the dam, and was built of uncoursed rubble masonry laid in a mortar made of one part Portland cement to three parts sand.

The concrete used in the facing of the dam and in the fore bay, power-house and machinery foundations, was made as follows: The mixture, by measure, was one part of Portland cement, three parts of sharp sand and four parts of broken stone. The broken stone was obtained by crushing granite, trap or gneiss boulders to cubes not exceeding 3½ ins., or smaller than 1 in. All materials used in this concrete were of the best quality; Alsen and Atlas cements were used exclusively and severe tests were made to insure uniformity in grade. No concrete was allowed to freeze before setting.

As shown in the cross-section, Fig. 1, the dam was carried below the level of the bed rock, in the center of the stream, to a distance of about 5 ft. This excavation was in one broad trench running entirely across the bed of the stream. The banks of the river are composed of solid sandstone and rise 25 to 30 ft. above the top of the dam; into these banks the dam is carried, 25 ft. on the north side and 18 ft. on the south side. The main seepage trench along the bottom of the dam was carried up and well into both banks, to cut off any chance of seepage around the ends of the dam, and all seepage trenches were filled and all exposed rock covered with concrete. The concrete facings, on the up and down stream faces and on the top of the dam, were well tamped in between the masonry and the wooden forms.

The dam is of the arched type in plan, Fig. 2, being built as the arc of a circle having a radius of 450 ft. Near the north bank, close to the stream-bed, two 5 ft. steel pipes were built into the concrete and fitted with gates and crabs, to permit of control of the water during construction, or time of repair; these would also regulate the flow over the spillway in time of flood. At the south end of the dam were "trash racks," and near them a sluice for disposing of debris and ice.

The length of the spillway is 108 ft. and the total length of the dam is 350 ft. The concrete aggregated 3,138 cu. yds., and the rubble masonry 5,025 cu. yds. This dam has experienced extremes of temperatures and stresses, and shows no cracks or signs of leakage. The work was started early in September, 1899, and about the middle of December the work was stopped by cold weather, with the north end completed, and the south end within 18 ft. of the top. On April 5th the laying of masonry and concrete was recommenced and completed about May 1. The work was then allowed to stand almost two months before the waste gates were closed down, and the water raised to the spillway. Figs. 3 and 4 show the dam in process of construction and completed.

THE FIRST DISCOVERY AND USE OF IRON ORE IN NORTH AMERICA.

The Old Colony Historical Society proposes, on July 10, 1901, to hold, at Taunton, Mass., a meeting commemorative of James and Henry Leonard, two of the first ironmasters in America. And in this connection the society has published a short historical sketch of the first attempt by Europeans to manufacture iron in what is now the United States.

The first mention of the existence of iron ore in this country is found in the report of the expedition fitted out by Sir Walter Raleigh, in 1585, and commanded by Ralph Lane. Lane and his men explored the country along the Roanoke, in North Carolina, and Thomas Harriot, the historian of the colony, says that in two places they found "the ground to be rockie, which, by the triall of a minerale man, was founde to hold iron richly." He suggests that it is "a good marchantable commoditie," as there is wood in abundance and labor cheap, for its reduction. But the colonists sought gold and not iron, no attempt was made to utilize this discovery; and in 1586 the adventurers returned to England.

In 1607 the first permanent English colony was founded at Jamestown, in Virginia, by the Virginia Company, of London; and on April 10, 1608, one of the company's ships sailed from Jamestown, loaded with iron ore, sassafras, cedar posts and walnut boards. This ship arrived in England on May 20, and Neill's history of the Virginia Company tells us that this iron ore was smelted and "seventeen tons of the metal were sold at £4 per ton to the East India Company." This was undoubtedly the first iron made from American ore. In 1619 the company sent to Virginia a number of persons skilled in the manufacture of iron, with instructions to "set up three iron works" in the colony. The first was undertaken that year and was located on Falling Creek, a tributary of the James River, about 7 miles below Richmond and 66 miles above Jamestown. Beverly, in his history of Virginia, says that this work was to be finished in 1621, but the death of three of the workmen delayed progress, and the Indian massacre of March 22, 1622, destroyed the works, and John Berkley, who was in charge, and all his workmen were slain. The revocation of the charter of the Virginia Company in 1624 disastrously ended this first attempt to make iron in America.

Though iron ore was first discovered in North Carolina, the first successful iron works were established in the Province of Massachusetts Bay. In 1632 Morton mentions the existence of "Ironstone" in New England; and in November, 1637, the general court of Massachusetts granted to Abraham Shaw one-half of the benefit of any "coles or yron stone" found in any common ground. In 1629 iron ore had been discovered on the upper Saugus River, near Lynn, and in 1642 speciments were taken to London by Robert Bridges and an attempt was made to form an iron company. As a result, The Company of Undertakers for the Iron Works was organized by eleven English gentlemen, who advanced £1,000

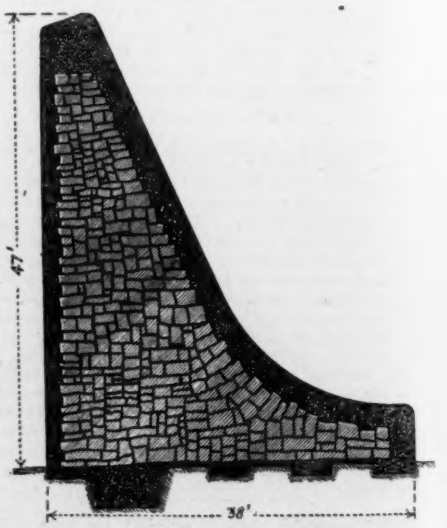


Fig. 1. Transverse Section of Concrete-Faced Dam for St. Croix Power Co., Wisconsin.

to start the works. John Winthrop, Jr., was interested in this scheme, and in 1643 he brought workmen and stock from England and began the erection of works on the western bank of the Saugus, and founded a village called Hammersmith. Large heaps of scoria, still existing, attest the working of this first iron making establishment in America. On May 14, 1645, the gen-

eral court passed an order declaring that "ye iron worke is very successful (both in ye richness of ye ore and ye goodness of ye iron)" and it referred to the furnace and "some tons of sowe iron cast in redines for ye forge." In 1648 Governor Winthrop, in writing to his son, says that "the furnace runs 8 tons per week, and their bar iron is as good as Spanish." But in 1688 these works seem to have been finally abandoned, after being fitfully operated from 1671 to that date. A small iron pot, of a capacity of about one quart, was the first article cast at these works by one Joseph Jenks. And in 1890 this relic was in the possession of his descendants, Llewellyn and Arthur Lewis, of Lynn, Mass.

This Joseph Jenks, in 1647, built a forge at the Lynn iron works for the manufacture of scythes and other edged tools. In 1652, he made at these works the dies for the "pine-tree shillings" coined by the colony; and in 1654 he made for the town of Boston the first fire engine built in America. He greatly improved the scythe then in use, and substituted for the short thick blade brought from England, a longer and thinner blade, strengthened by a square bar welded on the back. Jenks died in 1683.

Henry and James Leonard were also skilled workmen of Lynn, and originally forgemen, of Pontypool, Monmouthshire. These two men were ruling spirits in the third iron works in New England, the second enterprise established at Braintree, in 1643, being abandoned in 1647. The third works were located in the town of Taunton, two miles from the present city of Taunton, in 1652. The organizers employed the Leonards as practical workmen, and in 1653 the works were in operation. The family name of Leonard was connected with the Taunton Forge for many generations, and the old building was only demolished about a year ago, after a continuous existence of 244 years. The foundation wall alone remains of the oldest successful iron manufactory in New England.

Other early New England iron works are mentioned in the memoir. In 1668, Henry Leonard established iron works in Rowley village, 25 miles northeast of Lynn, but the works did not prove profitable, and Henry Leonard went to New Jersey. In 1677 an unnamed "iron works" was destroyed by the Indians, and about the same year iron was made at Topsfield, near Ipswich; and in 1680, at Boxford. The Whittington Iron Works, on Mill River, were established by James Leonard in 1670, and the works were in operation for more than one hundred years. In 1696-97 Thomas and James Leonard built the Chartley iron works, on Stony brook, within the limits of Taunton North Purchase. Other works are mentioned in Massachusetts.

In Connecticut, the first successful iron works seems to have been established in June, 1663, at New Haven. Rhode Island made iron certainly as early as 1675, when a forge was erected at Pawtucket by Joseph Jenks, Jr., son of Joseph Jenks, of Lynn. Iron does not appear to have been made within the limits of Maine, New Hampshire or Vermont, until the eighteenth century.

THE PROTECTION OF CITIES IN THE MISSISSIPPI VALLEY AGAINST THE ENCROACHMENT OF THE RIVER.

By L. W. Brown, M. Am. Soc. C. E.*

Insecure wharves and landings, due to constant change in the river banks, seriously interfere with the commerce and prosperity of cities located on the Mississippi River; and the existing conditions warrant the most thorough study of measures having for their purpose the greater security and permanence of these river fronts.

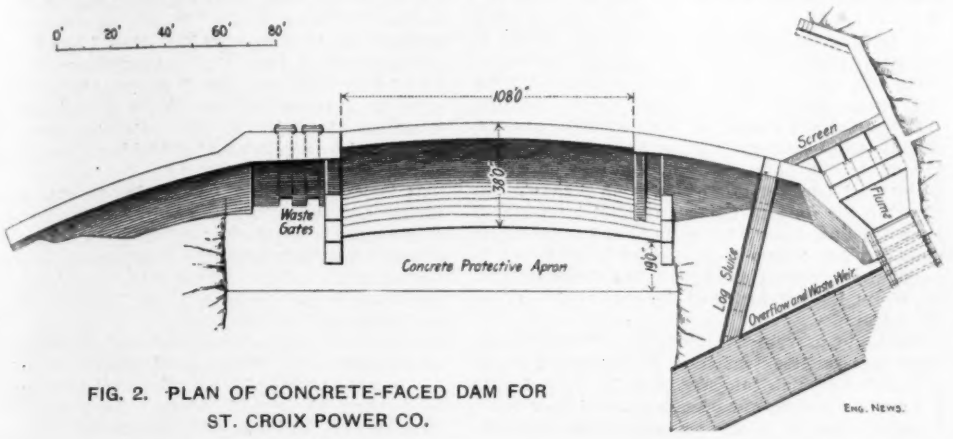


FIG. 2. PLAN OF CONCRETE-FACED DAM FOR ST. CROIX POWER CO.

Permanent wharves and landings not only attract commerce, but they also materially reduce the cost of handling goods and add to the profits of all concerned. The improvement of bad wharves is a necessity; for, otherwise, trade will be gradually deflected to other and better-equipped ports; even where the route to the improved landing may be less direct, and seemingly entails greater cost in transportation. As an illustration, we may take the port of New Orleans, and the case of a shipper of perishable goods, who is willing to erect suitable sheds or warehouses, adjacent to the landing in that city, for the protection of his goods. He finds that the water rates to New Orleans are considerably cheaper than to other ports, and he decides to take advantage of them. Upon investigation, however, he discovers that the banks of the river and the landings are not permanent at the point selected for the delivery of his property; and the buildings which he proposed to erect, at a cost of, perhaps, $30,000 to $40,000, may be in a few years, either in the river, or several blocks away from the river; either result rendering his investment valueless for the purpose intended. As a natural consequence of such a discovery, he will select another port, where stability of improvement is guaranteed, even if he has to pay an increased rate for transportation. Thus New Orleans, or any other port on the Mississippi River, will lose trade until measures are taken to insure permanence in its wharves and landings.

The cities themselves are deeply interested in securing their merchants and the owners of real estate against the possibility of direct injury to and the depreciation of valuable property. Under old conditions, expensive improvements, made in the interest of commerce, for manufacturing purposes, or even for residences, may either be engulfed in the river and land and improvements both lost; or they may be removed so far from the river front by reason of a change in the banks, that they no longer have the advantages of the location originally selected.

With the advance in engineering knowledge and experience, and with the appliances now available to engineers, there is no reason to doubt the efficiency of plans and the absolute permanence of works for bank protection, under the conditions stated. It only remains for the important cities on the Mississippi River to have their peculiar local needs carefully studied, and measures can then be adopted that will insure the requisite efficiency with the greatest economy possible.

The problem of maintaining permanent river banks has been before those interested for many years; and commission after commission has been appointed to devise ways and means; but no definite and satisfactory conclusion as to method of improvement has been reached by these bodies. This failure is probably due to the fact that before any valuable conclusion could be reached, very extensive study, investigation into existing conditions and experiment were absolutely essential; and for this work the time and necessary money were not available.

Up to the present time only two methods of protection have received any reasonable amount of study and actual experiment. These are the spur-dike and mattress method, adopted by the Mississippi River Commission twenty years ago, and still employed, notwithstanding the questionable success of this plan; and the pile bank-protection method, adopted four years ago and constructed by the Orleans Levee Board, in the City of New Orleans, under the recommendation and direction of the writer.

FIG. 4. VIEW SHOWING COMPLETED DAM FOR ST. CROIX POWER CO.

Before investigating the relative merits of these two systems, it is proper to study the cause of caving river banks and the conditions which increase the rate of caving, and which must be duly considered in making effective any plan having for its object the protection of the banks against abrasion.

*Engineer-in-Charge of Orleans Levee Board; Member of Advisory Board of Engineers to Sewerage and Water Board of New Orleans.

The velocity of an alluvial river, in certain places, may be retarded by a log, aggregation of floating material, or other cause; and while this retardation may be extremely small at the beginning, it allows part of the suspended matter to be precipitated, and it inaugurates the formation of a bar, or of a growing river bank. The deposits from precipitation constantly increase in volume, on the river bottom or on the banks, and the cross-section of the channel is thus reduced. This latter reduction increases the flood slope of the river, causing an accelerated velocity of flow. The river currents, following the laws of hydraulics, seek out and find the line of least resistance, and a new direction is given to the current, setting it towards the shore opposite to the one where the accretion is taking place. This new current impinges directly against the bank; very lightly at first; but it gradually increases in intensity until the slope of the bank is abraded and undermined, and serious caving follows.

If the bank against which the current is being gradually forced could be reinforced, so as to have sufficient resistance to overcome the action of the impinging current, and prevent abrasion of the safe slope of the bank, the current woud scour out the deposits which are gradually being formed; the proper cross-section would be restored and a stable channel would be secured.

It will be observed that the work required and the expense entailed in reinforcing a bank at the incipiency of the abrasion, is very limited, as compared with the outlay and intricate work demanded in holding a caving bank against further caving, with a heavy current setting directly against it. When not interfered with, the whole volume of the river will eventually strike against the abrading bank at right angles, as has often been observed. This condition requires a change in the direction of flow for the whole volume of the river; and demands an additional hydraulic head to overcome the loss of velocity caused by friction against the banks, and the friction of the water upon itself in turning this sharp angle.

It will be further observed, that when a bank has become so abraded as to practically change the direction of flow in the whole river, the action of the current upon the bank is so varied in its intensity throughout a given stretch which is failing by caving, that the contour of the bank becomes very irregular, and eddies, currents and cross-currents are created. Near the lower end of such an abrading bank, or where the current is forced to take another direction, whirlpools will be formed, which have enormous power, scour deep at points adjacent to the bank, and leave the slope of the bank very steep. If let alone these whirlpools gradually work their way farther in towards the bank, and constant disastrous caving follows.

To preserve a caving bank, or a bank against which an abrading current is impinging, it is a first essential to success, that provision be made for a uniform resistance throughout the stretch, and thus at once to remove the dangers resulting from the irregularities of the current. If we can guide the current past the abrading bank at a uniform velocity, the next step is to so plan our structure as to impede this velocity and allow deposits to form; and with this done any desired extension riverward can be gradually secured. Such a structure would provide a resistance more than equal to that which is required to force the current towards the opposite shore, and both this opposite shore and the channel of flow would in time be restored to their original position.

To be effective any such structure must extend into 30 or 40 ft. depth of water in the river and must reach above the level of the highest flood. The purpose of this structure is to regulate the velocity of flow by impinging against material provided; and we must effect the current all the way down to the bottom of the river, and by deflection retard its abrading action against the bank. Where such uniform impingement exists there will practically be a uniform velocity in the whole stretch. For a river flowing around a bend, against sloping banks, has its maximum velocity in the surface flow, the portion that has the greatest distance to travel, and this velocity decreases with the depth. Any structure that affects 30 or 40 ft. of the surface flow of a flood river, must necessarily affect the whole volume of the river. Experience has already shown that under the conditions named, accretion does take place, and the bank can be gradually extended riverward by pushing out the line of resistance against the impinging current.

This fact is very clearly and practically demonstrated by the gradual extension of the bank on the concave side of a bend in the river front of the City of New Orleans, between Calliope and Conti Sts., and the resultant abrasion of the opposite Algiers Point. Within the last 15 years, this concave bend, opposite Algiers Point, and between Calliope and Customhouse Sts., has been extended riverward fully 800 ft. by the erection of works which provide a uniform rate of impingement for the water passing the bend and thrown against the New Orleans shore. In this period the gradual abrading and caving of Algiers Point has been going on in the effort of the river to secure and maintain its proper cross-section of channel. As showing the gain to the commercial interests of New Orleans, in providing increased area on its most important river front, it may be said that 50 years ago this bend nearly reached Tchoupitoulas St., about 1,500 ft. inside of the present line of the river front.

Works to prevent the abrading of river banks must necessarily extend from the beginning of the caving far enough into the bend itself to deflect the current away from the shore; and they must be so planned as to actually eliminate the rough contours of the original bank, or provide conditions for accomplishing this end. The main purpose is to compel the current to move at a practically uniform rate, and to remove all cause for eddies, cross-currents and whirlpools.

In the method of protecting banks by spur-dikes and mattresses, the spur-dikes fail in their purpose, because it is the usual practice to place them directly in the apex of a caving bank, about 1,000 or 1,500 ft. apart. In this position they form a true dam; they decrease the cross-section of the river by the area of their sides exposed to the current, and in a dike 400 ft. long and 6 ft. high, this means 2,400 sq. ft. taken from this cross-section. These reductions in sectional area check the velocity of the volume of passing water until an increased head, directly over the dike, is provided sufficient to increase the velocity of flow and permit the given volume of water to pass through the contracted channel. This action interferes with the uniform velocity of flow and causes cross-currents, which set directly into and against the bank which the dikes are supposed to protect; these currents have sufficient force to abrade the banks and caving follows, both above and below the "protecting" structures. It may thus be said, that spur-dikes, placed in the lower Mississippi, positively accelerate, rather than retard, the caving of banks along whch they are placed.

Mattresses have failed in protecting abrading banks, for the reason that they are laid on the bank as it happens to exist, without any attempt being made to correct the rough contour of that bank. They thus fail to remove the damaging effect of eddies and cross-currents. Then, they do not extend sufficiently high to permit the surface flow of a flood river to impinge against them; and as a consequence the current often abrades the bank behind them, and the mattresses, bank and all go into the river. To be successful, mattress work—like any other protecting work, must begin at the point where the abrading begins and be laid continuously. But eevn if so laid, on a uniform contour, and high enough to intercept the surface flow of a flood river, mattresses do not admit of the building out of a concave bank; a procedure that is necessary if we wish to gradually lessen the intensity of the force of the current against the protecting work, and to gradually remove the necessity for any other structure than the natural bank which will be formed in time.

At New Orleans, mattresses were placed and replaced, between 1875 and 1882, between Marigny and Spain Sts.; but there was no cessation in the abrading of the banks and the caving of the pier-head wharves, until 1892, when a continuous wharf was built under the direction of the writer. Since 1892, no abrading or caving has occurred. Spur-dikes and mattresses were tried in the Third District bend, in the City of New Orleans, between the years 1888 and 1892; but instead of stopping the caving, it is not certain but that they accelerated it; and the caving continued very rapidly until 1897, when the only line of protection was a mud-box placed directly against highly-improved property. The former paved Front St., and the "battures" were entirely obliterated. The unanimous census of engineering opinion then was that it would be necessary to destroy this property and to construct a new levee 300 ft. back of the old line. The spur-dikes and mattresses evidently did not protect; and as a matter of fact, the river bank in 1897 was 75 to 100 ft. farther inshore than when the mattresses were laid. Figs. 1 and 2 show the position of two of these mattresses in this District in 1898; and Fig. 3 shows where a government mattress had been placed in 1896, and the condition of affairs in 1898, after the mattress, levee and a large piece of valuable railway property had gone into the river.

At Southport, in the Port of New Orleans, mattresses and spur-dikes had been put into position at great cost; but in September, 1900, caving occurred directly behind the location of the protecting works. This caving was of such an extent that—to prevent inundation, it was necessary to immediately construct a new levee; and incidentally, the extensive wharf and elevator improvements of the Yazoo & Mississippi Valley R. R. Co. were practically ruined, as all these and a large tract of valuable ground are now on the river side of the levee, and unprotected.

Directly after the extraordinary high water of 1897, the Orleans Levee Board was confronted with the extremely serious levee conditions in the Third District of the city, between Clouet and Lizardi Sts. For nearly a mile in length, as shown in Figs. 4 and 5, the protection of the city against inundation was a box-levee, 8 ft. wide, placed directly against highly-improved manufacturing and residence property; and the river had abraded the bank to such an extent that these mud-boxes were located directly upon the river bank. This condition prevailed, notwithstanding the fact that the U. S. Government had placed throughout this stretch very extensive works of the spur-dike and mattress type.

All the engineers, federal, state and city, united in recommending a new levee in the rear of the mud-boxes; and this recommendation was strenuously urged by the press and otherwise, notwithstanding the fact that such a measure would necessitate the destruction of a large amount of very valuable property, including factories that gave employment to 400 or 500 men. The Levee Board, in opposition to public opinion, determined to attempt to hold the then existing levee line by protecting works. It adopted the plans suggested by the writer and at once commenced work.

This method of Pile Bank Protection embraces three stages, with each stage executed as conditions may determine. The first stage is shown in Fig. 7, and includes the driving of 70 to 80-ft. piles throughout the stretch, at a uniform distance apart, and with their tops 3 ft. above the maximum flood heights. This work extended into the river to such a line as would best receive and guide the current past the abrading stretch; and provide a depth of 30 ft. or more of a flood river to impinge against these piles. The whole work was thoroughly braced and tied together, and the piles of the bulkhead row were closer together and supported a watertight, sheet-pile bulkhead. The latter formed support for a levee of greater dimensions than that of the old mud-box. As soon as these piles were in position the abrasion of the bank not only stopped, but accretion commenced, and the slope of the bank was materially improved; and the soundings show that this process of accretion is still going on.

In 1899, the conditions of the bank were such that the second stage in the operation was commenced. This stage is shown in Fig. 8; it consisted in removing the old mud-box; driving a new bulkhead on the river side of the one put in position in 1897, and the building of a substantial earth levee on the river side of Front St. This street was then reopened and paved, occupying territory covered by the river in 1897. It will be observed that the piles of 1897, on the line of the leveee, were cut off below the street surface, and additional piles were driven and the new levee was placed directly upon them; thus transmitting the weight of the levee to a plane on line with the

river bottom, and relieving the "batture" of this weight.

The third, or last, stage in this operation, is shown in Fig. 9. This contemplates the placing of a river slope of earth on the levee—as provided for in the second stage, and supporting this earth on piles. This work is to be done when the front bulkhead has deteriorated to such an extent as to require renewal; after this river slope is completed, a light wave-wash revetment would be put in place, as shown on the figure.

The advantages claimed for this system of pile protection are, that it extends continuously from the upper end of an abrading bank far enough into the bend to throw the current away from the shore. It extends sufficiently high above the highest flood line, and far enough out into the river to affect 30 or 40 ft. in depth of the river at high water. The piles are placed at uniform distances apart and provide a uniform obstruction to the current striking them. The front or river line of the work is placed on a line best adapted to receive and guide the current past the bank where abrasion was previously taking place. The whole theory of the success of work of this type is that a safe and uniform velocity is given to the river current; eddies, cross-currents, etc., are obliterated, and the uniform resistance of the piles to the flow checks the current sufficiently to allow suspended matter to be deposited on the bank previously abraded. Finally, the extent of the possible movement of the river bed towards the opposite shore is only limited by the gradual extension of the pile protection work into the river.

In comparing the two methods experimented with on the Mississippi River, the writer would not assume that either of the plans embraces all the requisite features for permanent bank protection. As can well be appreciated, the problem is many-sided, and of large proportions; but he believes that by continuous and careful study and experiments, methods will be developed which will be both economical and efficient; and that the Mississippi River, with all its vagaries and power, can be made, with absolute certainty, to follow the course laid down by the engineer.

HOLBROOK'S SPIRAL CURVES.

By E. Holbrook.*

About 21 years ago the writer found occasion to use transition curves on railroads, and found that the problem had not been worked out. This problem was to make a transition from circle to tangent such that the superelevation of the outer rail should at all points be proportionate to the centrifugal force. This could best be accomplished by passing from circle to tangent by a curve with a uniform rate of transition; the superelevation of the outer rail would then begin at the point of spiral and increase uniformly, reaching the maximum at the point of circle and remaining constant till the decreasing spiral was reached at the other end of the circle. This would give the simplest method for the trackman's use.

The nature of such a spiral is expressed by the equation

$$R\ L = A,$$

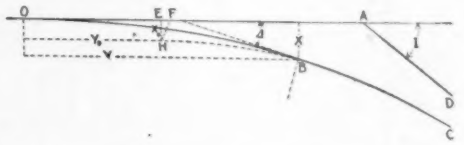
when R is the radius of curvature of the spiral at any point; L the length of the spiral measured from its origin, and A a constant which determines the rate of transition.

Having worked out the properties of such a spiral in connection with its relation to the adjacent tangent and circle and put it into use, the writer distributed among various assistant engineers of the "Pan Handle road," copies of the demonstration and tables, and from them it went to other roads. It was published briefly in "The Railroad Gazette" in December, 1880. The first engineer besides the writer to use these formulas extensively was Geo. W. Kittridge, now Chief Engineer of the "Big Four," who put spiral ends on many of the curves of the "Pan Handle," both east and west of Steubenville, O.

Since that time volumes have been written on the subject and many tables arranged, with countless variations, to suit the ideas of engineers as to how best to lay out this curve. They are all, however, but different aspects or parts of what was contained in the original work, and have been made with one or more of three objects in view, viz.: to devise a scheme where tables could be dispensed with; to overcome a fancied want of elasticity, or to escape a little higher mathematics which enter into the demonstration but not into the application.

*Chief Engineer Kansas City Southern Ry. Co., Kansas City, Mo.

By the help of the following figure and table the actual work in the field is briefly explained:



Let O A and A D be two tangents, intersecting at the point A, making the angle I.

O B is the spiral connecting the circle H B C with the tangent O A, having a length O E equal L.

$\triangle$ is the total curvature of the spiral.

d is the deflection angle to be turned off from the tangent to locate any point, as B.

To locate the point O for any given degree of curve and intersection angle I, we have: $O\,A = T = A\,F + F\,E + E\,O = R \text{ tang } \frac{1}{2}\,I + X_o \text{ tang } \frac{1}{2}\,I + Y_o$, or $(R + X_o) \text{ tang } \frac{1}{2}\,I + Y_o$.

If we wish to lay out the spiral with a rate of transition of one degree in 60 ft., enter the Table I at the line beginning with the required degree and take out the value of R, to which add by inspection the value of X_o found on the same line and multiply the sum by tang ½ I, as in ordinary circular curves; to this product add the value of Y_o found on the same line, and you will have the length of tangent for locating O. Set the transit at O and turn off from the tangent the deflections found in the table for various lengths of spiral for locating the spiral until the required degree is reached, as shown by the figures in the first column. Then set up at B, backsighting on O, and turn off twice the total deflection of the spiral, and the line of sight will then be on the common tangent of spiral and circle. The circle can then be produced in the usual manner.

Putting in true transition curves adds practically nothing to the work except as they make the curves longer. They can be put in as quickly by one who is familiar with them as any approximate method that is not rank fudging. There are no offsets to make, no parallel tangents, no curve to run that is not used and no stake to drive that is not a center stake, and stakes may be driven at any station and as few or as many as desired.

As to the elasticity of this method, 20 years' experience has demonstrated that a single rate of transition should be used on new work, and it is very rare that more than two are required on old work. If we use two rates, viz.: one degree in thirty ft. and one degree in sixty ft., we can relocate curves up to 6° without getting the stakes outside of the rails; but if some other rate is desired, say one degree in twenty ft., inspect the table. Table No. I. has a rate of one degree in sixty ft., and No. II. has one degree in thirty ft. Compare the value of d on corresponding lines of the two tables and note that one is just double the other, or vary as the rate of transition, so we can lay out the desired spiral by adding 50% to the values of d found in Table No. II. Note that the values of X_o, on lines beginning with the same degree, are as 4 to 1, or vary inversely as the squares of the rates of transition, which enables us to find the required value of X_o with very little work. Observe that Y_o may be obtained by subtracting a small quantity from ½ of L; and observe that this quantity varies in the tables as the cubes of the rates of transition, which enables us to find Y_o in case the spiral is so long that it cannot be seen by inspection.

Many have objected to the use of the true transition curve because its demonstration requires some higher mathematics. The objection is no better founded than an objection to a table of sines and cosines, because they require the same. A wide experience has demonstrated that young engineers with only a rudimentary knowledge of trigonometry can grasp the general idea and learn to apply it as easily as they can learn to solve a triangle.

At the time that the original demonstration was made the writer did not attempt to ascertain the relation directly between $\triangle$ and d, and did not observe for a time that to the limit of the table, $d = \frac{\triangle}{3}$ and consequently that the angle at B turned off to get on the common tangent was twice d, that is $\triangle - d = 2d$. This ratio does not hold for large values of $\triangle$, which goes on increasing indefinitely, while d never reaches 65°.

No case has yet occurred to the writer where he could not stake out the whole spiral at one setting; but if the case arises the transit can be moved along the tangent and the remaining deflection easily calculated by use of the values of X and Y found in the tables. Usually, a table like the one given below supplies all that is required; $\triangle$ being obtained if required by multiplying d by 3.

(1° = 60 ft.)

Degs.	L.	X.	Y.	Y_o	d.
0° 20′	20	0.01	20.00	10.00	0° 00.7′
0° 40′	40	0.03	40.00	20.00	0° 02.7′
1° 00′	60	0.10	60.00	30.00	0° 06.0′
1° 20′	80	0.25	80.00	40.00	0° 10.7′
1° 40′	100	0.48	100.00	50.00	0° 16.7′
1° 50′	110	0.64	110.00	55.00	0° 20.2′
2° 00′	120	0.84	120.00	60.00	0° 24.0′
2° 10′	130	1.06	129.99	65.00	0° 28.2′
2° 20′	140	1.33	139.99	70.00	0° 32.7′
2° 30′	150	1.64	149.98	75.00	0° 37.5′
2° 40′	160	1.99	159.98	80.00	0° 42.7′
2° 50′	170	2.38	169.97	85.00	0° 48.2′
3° 00′	180	2.83	179.96	90.00	0° 54.0′
3° 10′	190	3.33	189.95	95.00	1° 00.1′
3° 20′	200	3.89	199.93	99.99	1° 06.7′
3° 30′	210	4.49	209.91	104.99	1° 13.5′
3° 40′	220	5.16	219.89	109.98	1° 20.7′
3° 50′	230	5.90	229.86	114.98	1° 28.2′
4° 00′	240	6.70	239.83	119.97	1° 36.0′
4° 10′	250	7.58	249.79	124.97	1° 44.2′

If it is desired to use a subchord, or a degree of curvature between those given in the tables, d for the subchord can easily be found by remembering that d increases as the square of the length, X_o and Y_o change so slowly that the former may be obtained by second differences and the latter from first differences, as found in the table. For curves of small total curvature it is sometimes best to make the whole curve a transition. In such case the total curvature of each spiral will be $\triangle = \frac{1}{2}\,I$, and the length of the tangent will be $y + x \text{ tang } \frac{1}{2}\,I$. If the rate of transition in the table does not give an external distance to suit the location, find the ratio of the one given in the table to the one desired and increase or decrease the length of the tangents and the chords used in staking out accordingly, since all such curves between tangents of a given intersection angle are similar.

For the external distance E we have

$$E = x \sec \tfrac{1}{2}\,I.$$

It is sometimes necessary to put in a circular curve with transition that will have a given external distance, I being given. Find the degree of a simple curve having the required external distance, then

Let

E′= the required external distance,

D = the degree of a simple curve with external E,

D′= the degree of curve with transition and external E′,

$$D' = D\left[\frac{E + X_o \sec \tfrac{1}{2}\,I}{E}\right],$$

where X_o corresponds to D′; but X_o changes so slowly and the difference between D′ and D is not usually over 10 minutes, so the value of X_o can be estimated sufficiently close to get the value of D′ at the first approximation.

If it is desired to find E for any given value of R and I we have

$$E = R \text{ tang } \tfrac{1}{4}\,I \text{ tang } \tfrac{1}{2}\,I + X_o \sec \tfrac{1}{2}\,I.$$

A large number of special cases of compound and reverse curves with transitions will be found worked out in the appendix to the Ohio State Railway Commission's report for 1884. However, keeping in mind that in all cases the circle is removed a distance X_o from the tangent, and in compound curves the two circles stand in the same relation to each other that the tangent and circles do in the ordinary case, no difficulty will be experienced.

TABLE I.—(1° = 60 ft.)
(Spiral increasing 0° 01′ per ft.)

Degrees.	Radius, ft.	L.	△	X.	Y.	X_0.	Y_0.	d.
0° 20′	17188.75	20	0° 02′	0.01	20.00	0.00	10.00	0° 00.7′
0° 40′	8,594.37	40	0° 08′	0.03	40.00	0.01	20.00	0° 02.7′
1° 00′	5,729.60	60	0° 18′	0.10	60.00	0.03	30.00	0° 06.0′
1° 20′	4,297.15	80	0° 32′	0.25	80.00	0.07	40.00	0° 10.7′
1° 40′	3,437.75	100	0° 50′	0.48	100.00	0.11	50.00	0° 16.7′
1° 50′	3,125.21	110	1° 00½′	0.64	110.00	0.16	55.00	0° 20.2′
2° 00′	2,864.80	120	1° 12′	0.84	120.00	0.21	60.00	0° 24.0′
2° 10′	2,644.41	130	1° 24½′	1.06	129.99	0.27	65.00	0° 28.2′
2° 20′	2,455.52	140	1° 38′	1.33	139.99	0.32	70.00	0° 32.7′
2° 30′	2,291.82	150	1° 52½′	1.64	149.98	0.42	75.00	0° 37.5′
2° 40′	2,148.57	160	2° 08′	1.99	159.98	0.52	80.00	0° 42.7′
2° 50′	2,022.20	170	2° 24½′	2.38	169.97	0.60	85.00	0° 48.2′
3° 00′	1,909.85	180	2° 42′	2.83	179.96	0.71	90.00	0° 54.0′
3° 10′	1,809.20	190	3° 00½′	3.33	189.95	0.84	95.00	1° 00.1′
3° 20′	1,718.80	200	3° 20′	3.89	199.93	0.99	99.99	1° 06.7′
3° 30′	1,637.01	210	3° 40½′	4.49	209.91	1.13	104.99	1° 13.5′
3° 40′	1,562.60	220	4° 02′	5.16	219.89	1.28	109.98	1° 20.7′
3° 50′	1,494.66	230	4° 24½′	5.90	229.86	1.48	114.98	1° 28.2′
4° 00′	1,432.39	240	4° 48′	6.70	239.83	1.67	119.97	1° 36.0′
4° 10′	1,375.09	250	5° 12½′	7.58	249.79	1.90	124.97	1° 44.2′
4° 20′	1,322.20	260	5° 38′	8.51	259.75	2.13	129.97	1° 52.7′
4° 30′	1,273.20	270	6° 04½′	9.53	269.70	2.38	134.97	2° 01.5′
4° 40′	1,227.76	280	6° 32′	10.63	279.64	2.66	139.96	2° 10.7′
4° 50′	1,185.40	290	7° 00½′	11.81	289.57	2.96	144.94	2° 20.1′
5° 00′	1,145.91	300	7° 30′	13.07	299.49	3.26	149.92	2° 30.0′
5° 10′	1,108.95	310	8° 00½′	14.42	309.40	3.61	154.91	2° 40.2′
5° 20′	1,074.28	320	8° 32′	15.86	319.29	3.97	159.89	2° 50.7′
5° 30′	1,041.73	330	9° 04½′	17.39	329.17	4.34	164.87	3° 01.5′
5° 40′	1,011.09	340	9° 38′	19.02	339.04	4.76	169.85	3° 12.7′
5° 50′	982.21	350	10° 12½′	20.74	348.89	5.19	174.82	3° 24.2′
6° 00′	954.93	360	10° 48′	22.56	358.72	5.65	179.79	3° 36.0′

TABLE II.—(1° = 30 ft.)
(Spiral increasing 0° 02′ per ft.)

Degrees.	Radius, ft.	L.	△	X.	Y.	X_0.	Y_0.	d.
0° 20′	17188.75	10	0° 01′	0.00	10.00	0.00	5.00	0° 00.3′
0° 40′	8,594.37	20	0° 04′	0.01	20.00	0.00	10.00	0° 01.3′
1° 00′	5,729.60	30	0° 09′	0.03	30.00	0.01	15.00	0° 03.0′
1° 20′	4,297.15	40	0° 16′	0.06	40.00	0.02	20.00	0° 05.3′
1° 40′	3,437.75	50	0° 25′	0.12	50.00	0.03	25.00	0° 08.3′
2° 00′	2,864.80	60	0° 36′	0.21	60.00	0.05	30.00	0° 12.0′
2° 20′	2,455.53	70	0° 49′	0.33	70.00	0.08	35.00	0° 16.3′
2° 40′	2,148.59	80	1° 04′	0.49	80.00	0.13	40.00	0° 21.3′
3° 00′	1,909.86	90	1° 21′	0.70	90.00	0.17	45.00	0° 27.0′
3° 20′	1,718.89	100	1° 40′	0.97	99.99	0.24	50.00	0° 33.3′
3° 40′	1,562.61	110	2° 01′	1.29	109.99	0.31	55.00	0° 40.3′
4° 00′	1,432.39	120	2° 24′	1.67	119.98	0.41	60.00	0° 48.0′
4° 20′	1,322.21	130	2° 49′	2.12	129.97	0.52	65.00	0° 56.3′
4° 40′	1,227.76	140	3° 16′	2.65	139.95	0.68	70.00	1° 05.3′
5° 00′	1,145.91	150	3° 45′	3.26	149.94	0.81	74.99	1° 15.0′
5° 20′	1,074.29	160	4° 16′	3.96	159.91	0.98	79.98	1° 25.3′
5° 40′	1,011.10	170	4° 49′	4.75	169.88	1.18	84.98	1° 36.3′
6° 00′	954.93	180	5° 24′	5.64	179.84	1.40	89.97	1° 48.0′
6° 20′	904.67	190	6° 01′	6.63	189.79	1.65	94.97	2° 00.3′
6° 40′	859.44	200	6° 40′	7.74	199.73	1.93	99.96	2° 13.3′
7° 00′	818.52	210	7° 21′	8.95	209.66	2.23	104.94	2° 27.0′
7° 20′	781.30	220	8° 04′	10.30	219.57	2.57	109.93	2° 41.3′
7° 40′	747.33	230	8° 49′	11.76	229.46	2.93	114.92	2° 56.3′
8° 00′	716.20	240	9° 36′	13.37	239.33	3.34	119.89	3° 12.0′

For the benefit of those who may care to look into the mathematical properties of the spiral, I append the following:

(1.) $$R\ L = A.$$

For a transition af the rate of 1° in 60 ft., we have

$$A = 343{,}775\ .\ R = \frac{343{,}775}{L}.$$

Since the curvature at any point is twice the average curvature between the point and the origin we have the total curvature.

(2.) $$\triangle = \frac{L}{2} \times \frac{L}{100} = \frac{L^2}{200} \text{ (in minutes)};$$

or

$$\triangle = 0.0000014545\ L^2 \text{ (in terms of arc radius unity)}.$$

Let

$$0.0000014545 = a, \text{ then } \triangle = a\ L^2.$$

By inspection of figures we see

(3.) $$\frac{d\ y}{d\ L} = \cos \triangle = \cos a\ L^2, \ \therefore$$

$$Y = \int_0^L \cos a\ L^2\ d\ L = L - \frac{12\ a^2\ L^5}{1.2.3.4.5.} + \frac{1680\ a^4\ L^9}{1.2.3.4.5.6.7.8.9.} + \&c.$$

(4.) $$\frac{d\ x}{d\ L} = \sin \quad = \sin a\ L^2,$$

$$\therefore\ X = \int_0^L \sin a\ L^2\ d\ L = \frac{2\ a\ L^3}{1.2.3.} + \frac{1200\ a^3\ L^7}{1.2.3.4.5.6.7.} + \&c.$$

or, putting in the value of a and reducing we have

(5.) $$Y = L - 0.000000000000211557\ L^5 + \&c.$$

(6.) $$X = 0.0000004848\ L^3 + \&c.$$

(7.) $$X_0 = X - R \text{ versin } \triangle$$

(8.) $$Y_0 = Y - R \sin \triangle$$

(9.) $$\tan d = \frac{X}{Y}, \text{ from which value of d can be computed or dividing equation (6) by equation (5), substituting in equation for value of tan } \triangle, \text{ and reducing we have}$$

(10.) $$d = \frac{L^2}{600} - \&c.;$$

but

(11.) $$\triangle = \frac{L^2}{200},$$

within the range of practice we may use

(12.) $$d = \frac{\triangle}{3}.$$

Likewise

$$X_0 = \frac{X}{4} - \&c.$$

A COUPLING FOR THIN COPPER PIPES.

The accompanying drawings, reproduced from our Paris contemporary "Le Genie Civil," show a design of coupling for copper pipes to be used in place of riveted or brazed joints. As will be observed, the coupling consists of two metal collars, one with a rectangular and one with a conical inner face, and of a round wire ring, which are drawn together by bolts so as to clasp and bring

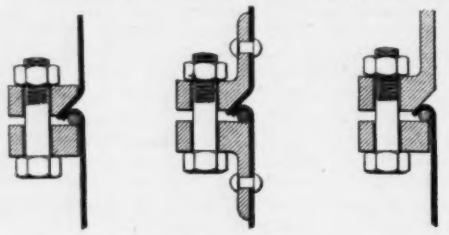


Fig.1. Fig.2. Fig.3.

A Coupling for Copper Pipes.

together the two ends of the pipe, as shown. Fig. 1 shows the simplest form of this coupling. Fig. 2 is the same coupling with the collars provided with flanges to which the ends of the pipes may be riveted to secure a stronger joint. Fig. 3 shows the coupling used to join a copper pipe to another of iron, or to the reservoir with which it connects.

THE ENGLISH LAW OF UNDERGROUND WATER.

Some recent English law decisions, relating to the interception of the underground flow of water, are noted in London "Engineering." As the ever-increasing demand for water to supply large towns makes this question an important one, an abstract here follows:

The first case is that of Kibble vs. the Chipping Norton Urban District Council, in which the plaintiffs were the owners of a mill, with the wheel turned by water from the small River Glyme. The spring forming the source of this stream was on a hillside, a little below an outcrop of the Blue Lias, with the dip of the formation towards the valley. Above the Lias was a bed of broken Oolitic limestone, the exact nature of which was disputed by the geological experts. The water supplying the spring percolated through this latter formation.

The defendants, in order to augment their water supply, had dug a trench in this Oolite, 360 ft. long and 14 ft. deep; and in it laid a pipe leading to the storage tank. The plaintiffs claimed that the water did not "percolate" through this limestone formation, but came through comparatively large channels formed by the carbonic acid in solution in the water dissolving the limestone and creating cavities in the rock. It was argued that the defendants had no right to interfere with water flowing in defined channels. The defendants, on the other hand, proved by the engineer and others that no defined flow of water was ever cut by their trench; and that the water "sweated" out of the sides all over, and that the limestone was in moderate pieces embedded in the sands.

Mr. Justice Cozens Hardy gave judgment for the defendants, on the ground that the water merely percolated and did not flow in defined channels.

This decision is in accordance with the existing English law relating to underground water, as passed upon in Chasemore vs. Richards, generally regarded as the leading case. Here, a land owner and mill owner had enjoyed the use of a stream for 60 years, which was chiefly supplied by percolating underground water. He lost the use of this stream when an adjoining land owner dug on his own ground an extensive well for the purpose of supplying water to the inhabitants of the district, many of whom had no title, as land owners, to the use of the water.

The Chief Justice ruled that the mill owner had no right of action. He said: "We think the present case is not to be governed by the law which applies to rivers and flowing streams, but that it rather falls within the principle which gives to the owner of the soil all that lies beneath its surface; that the land is his property, whether it is solid rock, or porous earth, or venous earth, or part soil and part water; that the person who owns the surface may dig therein and apply all that is there found to his own purposes at his free will and pleasure; and that if, in the exercise of such right, he intercepts or drains off the water collected from underground springs in his neighbor's well this inconvenience to his neighbor falls within the description of *damnum absque injuria* which does not become the cause of action."

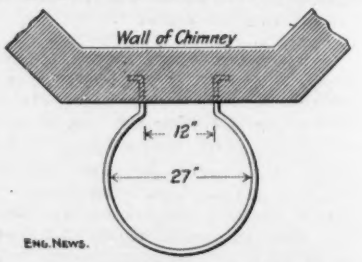
Our contemporary, in noting this decision as recognized law, presumes that the courts refuse to recognize any right in percolating water because the course of such water is undefined, and any attempt to trace it would lead to protracted litigation. And further, if such right were recognized a man might be prevented from intercepting rain water by conveying it from a roof to a tank; because, if not thus intercepted, it might have entered the ground and fed a neighbor's well. It is a little hard, as "Engineering" says, that this law should permit the interference with natural underground flow for the purposes of a public water supply; where a mill owner is deprived of a long established source of supply for the benefit of those who reside at a distance and have no rights in the land from which the water is taken.

A SAFE LADDER FOR CHIMNEYS, STAND-PIPES OR OTHER HIGH STRUCTURES.

We take the following from our contemporary, "The American Machinist:"

Wm. B. Cogswell, vice-president and chief engineer of the Solvay Process Co., of Syracuse, N. Y., relates that while traveling through Germany a short time since he caught a glimpse from a car window of a brick smokestack that was being erected and saw upon it a ladder of new design, the advantages of which immediately impressed him very strongly, with the result that a new smokestack which has since been built in his own establishment in Syracuse has this ladder applied to it.

The rungs of this ladder are made as shown by the accompanying sketch made by Mr. Cogswell. The idea is that a man climbing to the top of this stack gets inside of these rungs, where he is much more secure than upon the ordinary ladder, as he cannot be blown off and feels much



A Safe Chimney Ladder.

more safe than when he is outside, besides which he can at any time rest himself completely by standing with both feet and his elbows upon the rungs of the ladder at the same time, or he can put his arms over the rungs at any time, or if he likes can have a piece of plank 30 ins. long with him, which he can put across one of the rungs and stand upon, thus making himself perfectly secure while working.

It is Mr. Cogswell's belief, and we agree with him, that all chimney ladders should be made in this way.

While the cut shows the ladder applied to a brick chimney, there is no reason why similar ladders should not be applied to steel plate chimneys, water-works stand-pipes, or any other lofty structures to which ladders are attached.

BOOK REVIEWS.

WATER-WORKS DISTRIBUTION.—A Practical Guide to the Laying Out of Distributing Mains for the Supply of Water to Cities and Towns. By J. A. McPherson, Assoc. M. Inst. C. E. Cloth; 6 × 8 ins.; pp. 154; 20 plates and 103 illustrations in the text. London: B. T. Batsford. New York: D. Van Nostrand Co. 6 shillings in England; $2.50 in America.

This book is more comprehensive than might be supposed from its title page. It gives some attention to service reservoirs, standpipes and water towers, and a considerable amount of space to the various plumbing fixtures through which water is delivered. There is also a special chapter on water meters and allied subjects. Standpipes, as here used, are vertical pipes of the same diameter as the horizontal main with which they are connected. The whole conduit system of a water-works plant is discussed, including aqueducts or supply mains at one extreme, and service pipes at the other. The design of pipe systems to meet various local conditions is taken up in

some detail, and the hydraulics of the subject is not neglected. An average daily consumption of about 30 (U. S.) gallons, with a maximum of 60 gallons, is considered ample, while fire hydrants on 3-in. laterals are condemned. These two points illustrate the vast difference between the amount of water consumption in England and America. An injector hydrant is described, by means of which the high pressures in the hydraulic power mains of some English cities may be utilized to increase the height to which streams from the ordinary water mains may be thrown. A detail of water-works practice not so advanced as the foregoing is a description of wooden fire plugs, driven into a cast and bored right-angled branch in a street main. The plugs are loosened by hand and removed by the water pressure, and a "standpipe" inserted temporarily when it is desired to use the plug. Although the author strongly recommends the use of fire hydrants instead of fire plugs it seems strange that such a primitive device should be still used in any country to such an extent as to warrant an illustrated description in a book devoted wholly to present water-works practice. It may be added that this device throws light upon the use of the term fire plug instead of fire hydrant, still common in some sections of America. It would be interesting to know how many years it has been since a wooden fire plug, like those described in this book, was used in an American city or town, and more interesting still to know how long it has been since one has been included in the construction of a water-works plant.

The book as a whole treats of the design of distribution systems, rather than their construction, and contains little relating to operation, unless it be the chapter on meters and other recording apparatus. The author states that he has "had upwards of 24 years' experience in water-works distribution." Certainly he has written a practical handbook along the lines already indicated. It might well be added to the library of every water-works engineer and superintendent who wishes to keep posted on "how others do it" and would also prove useful to beginners in water-works engineering and to many water-works superintendents who are not engineers.

A MANUAL OF HYGIENE AND SANITATION.—By Seneca Egbert, A. M., M. D., Professor of Hygiene in the Medico-Chirurgical College of Philadelphia. New (2d) and revised edition. Cloth; 6 × 8 ins.; pp. 427; 77 illustrations. Philadelphia and New York: Lea Brothers & Co. $2.25.

This volume appears to have been designed primarily as a text book for medical students. As such, and as an elementary work for general use, it is remarkably sane and satisfactory. Most physicians fail to grasp the real significance of municipal sanitary problems, largely because they have had no adequate instruction on the subject in their medical schools and have been too busy to work up the subject in after years. The author of this book makes but few of the slips common to medical writers when dealing with water supply, sewerage, sewage and garbage disposal. We note that he includes scarlet fever and diphtheria among the diseases sometimes conveyed by water, but does not specify any instance of the kind. It is doubtful if any could be given. We also note that he says a sewage effluent from a chemical precipitation plant may be filtered at a daily rate of 1,000,000 to 2,000,000 gallons an acre, which is pretty high. But such things may be passed by in the case of a medical writer who does not dilate upon the spread of disease germs by so-called sewer gas, and by decaying garbage. More emphasis might well have been laid on the spread of typhoid fever, scarlet fever and diphtheria by milk, and on the importance of close sanitary supervision of the milk supply to guard against epidemics of those diseases. Among the subjects to which separate chapters are given these may be mentioned: Bacteriology, air, ventilation and heating, water, the removal and disposal of sewage, and disinfection. A chapter on "Military Hygiene" has been written for the present edition. The first edition appeared in 1898.

RESERVOIRS FOR IRRIGATION, WATER POWER AND DOMESTIC WATER SUPPLY.—With an Account of Various Types of Dams and the Methods and Plans of Their Construction, Together with a Discussion of the Available Water Supply for Irrigation in Various Sections of Arid America; the Distribution, Application, and Use of Water; the Rainfall and Run-off, the Evaporation from Reservoirs; the Effect of Silt Upon Reservoirs, Etc. By James Dix Schuyler, M. Am. Soc. C. E., M. Inst. C. E., New York: John Wiley & Sons. London: Chapman & Hall. Cloth; 7 × 10 ins.; pp. 414; 208 illustrations. $5.

Engineers engaged in any of the various branches of water-supply engineering, whether for irrigation, power or city purposes, have reason to be glad that Mr. Schuyler has revised, extended and made generally available the excellent paper on "Reservoirs for Irrigation," which he wrote for the eighteenth annual report of the U. S. Geological Survey. The paper named was printed separately, but the edition was soon exhausted.

The plan of this volume is simple and effective. There is a chapter, each, on rock-fill, hydraulic-fill, masonry and earthern dams; one on natural reservoirs and another on projected reservoirs. An appendix contains tables of possible reservoir areas and capacities on sites in the West, selected by the U. S. Geological Survey. There are numerous reproductions of both line drawings and handsome photographs.

Aside from brief statements of the principles involved in the design and construction of each type of dams, and criticisms of the various structures reviewed, the book, in the words of Mr. Schuyler,

is chiefly a straightforward description of those works with which the author has become familiar, either as consulting engineer, or as designer and constructor, or merely as an interested observer of the work of other engineers.

When well presented, as this one is, books of the kind described often form the most useful sort of technical literature.

ELABORATION DES METAUX DERIVES DU FER, REACTIONS METALLURGIQUES.—Par L. Gages, Capitaine d'artillerie. Paris: Gauthier-Villars, Quai des Grands-Augustins 55. Paper; 4⅝ × 7⅝ ins.; pp. 175; illustrated. 2.50 francs.

The author presents the theoretical principles upon which rests the reduction of metals derived from iron ore; and he first sets forth the characteristics of metallurgical reactions, and the processes of refining metals. Taking up the old puddling methods, he proceeds to the more modern methods of treating metals, as introduced by Bessemer and Martin-Siemens, whereby, instead of producing metal in a pasty state, it is utilized in a liquid state and in great volume, comparatively, and poured directly into the ingot molds. He takes up the problem of the elimination of phosphorus, the refining of various special steels, and their cementation, under conditions in which the carbon is no longer the principal modifying element of the properties of the iron.

A PRACTICAL TREATISE ON SEGMENTAL AND ELLIPTICAL, OBLIQUE OR SKEW ARCHES.—Setting forth the principles and details of construction in clear and simple terms. By George Joseph Bell, County Surveyor and Bridge Master, of Cumberland, England. Sold in New York by Spon & Chamberlain, 12 Cortlandt St. Cloth; 7½ × 10⅝ ins.; pp. 125; illustrated. Price, $8.40.

This is the 1896 edition of this work, already reviewed by this journal in our issue of Oct. 22, 1896. But it is supplemented by the insertion of a series of nine plates, showing photographic views of the Sickergill oblique arch in its progressive stages of construction, and under each view is additional text explaining each stage of construction; the templates used in arriving at given results, and their method of application. These views and the accompanying explanatory matter to a very considerable extent remove the objections made in our previous review—in which we noted the absence of sufficiently detailed illustrative matter relating to the construction of the templates and their application in stone-cutting. By referring to them the text proper is made more clear—especially to the novice in skew-arch construction. It is only due to the author to say that his general text is devoid of the mathematics too often found in the older works upon this subject; and he has described his methods in detailed and clear terms. The mathematics involved in oblique arch construction are treated in a separate chapter, which is followed by examples of these arches, for different spans and skews, worked out in detail. We can only repeat, that this work is undoubtedly the best, as it is the latest treatise upon this subject.

VOCABULAIRE, FRANCAIS, ANGLAIS, ALLEMAND.—Techique, industriel et commercial, etc. Par E. Hospitalier, Ingenieur des Arts et Manufactures, Professeur a l'Ecole de Physique et de Chimie industrielles de la Ville de Paris, Redacteur eu chef de "L'Industrie Electrique." Paris: Published by "L'Industrie Electrique," 9 Rue de Fleurus. Cloth; 3½ × 8 ins.; pp. 316.

This technical, industrial and commercial dictionary in three languages is arranged on a somewhat original plan. The author, while not claiming that it is as yet a full list of foreign technical terms, believes that he has produced a solid foundation upon which a more complete dictionary can gradually be built up. The peculiarity of the volume lies in the fact that the English, French and German terms are arranged under a single alphabetical order and not with each language separate as usual. Opposite each word in the alphabetical range are its synonyms in the two other languages; the words themselves are printed in a black-face type; the English equivalents are in italics; the French in small capitals, and the German in ordinary light-face Roman type. By this arrangement any technical term contained in this book in either English, French or German can be at once found by looking in the proper alphabetical order for this word; and opposite to it will be found equivalents in two other languages. The result is a compact volume, very easy of reference, and a very useful aid to the engineer or commercial man who has to wrestle with the intricacies of a foreign tongue.

THE SOUCHIER BINOCULAR RANGE-FINDER.

The range-finder here described is a French invention, primarily intended for military use and so adjusted as to enable an observer to closely approximate the distance away of a man on foot or on horseback. It could also be utilized, however, in engineering reconnoisance work by sighting upon a man; or, better still, upon a rod of known height divided into three or four distinct divisions of color, so that the relative positions of the superimposed images could be more accurately fixed.

In construction the apparatus consists of two parallel telescopes, like an ordinary field glass, fitted with "long-range" lenses, and connected together by two bars, which are arranged to slide one within the other so as to adjust the parallel lines of sight to the individual width between the eyes of the observer. The focusing is done in the ordinary manner, and the apparatus will serve all the purposes of a first-rate field-glass.

At the eye-end of the telescopes, however, are two small pivoted levers, each carrying at its end a double refracting quartz prism. These prisms can be thrown into the lines of sight and they then have the effect of producing two images in the field of vision, superimposed, or standing vertically one above the other. The distance is estimated by observing how much one image covers the other; and a scale is arranged by sighting upon a man or a horseman stationed at known distances from the observer. Diagrams of a foot soldier and of a cavalryman, with scales attached, are printed on the barrels of the telescope for easy reference, and are protected by the sun-shades at the object ends of the glasses. One of these diagrams is reproduced herewith.

To use the instrument, it is first adjusted to the proper width between the eyes, and then focused upon the object. This done, the two prisms are turned into the line of sight, and the two images then appear. If the head of the lower man—for example—touches about the middle of the upper image, an inspection of the man-diagram shows that the object is 700 m., or about 2,300 ft. distant. If the image of one man seems to stand directly upon the head of the other, the distance is 1,400 m., or 4,592 ft. In the case of the cavalryman-

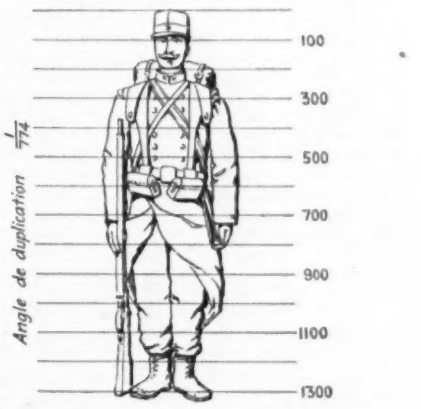


Diagram Attached to Range-Finder Field Glass

diagram the same arrangement of images as the last would call for a distance of 2,000 m., or 6,560 ft.

A personal examination of the instrument shows that this duplication of images naturally diverts some of the light and decreases somewhat the clearness of the images. It is also essential to the accuracy of the estimate of distance that the image be as near the center of the field as possible. With a little practice in its use, however, we judge it would be easy to become quite expert in making rapid measurement of distances. The greatest field of usefulness of the instrument is evidently in military service. The usefulness of a range-finder which is no more complicated or liable to get out of order than an ordinary field-glass, and which gives at once the distance away of either infantry or cavalry is obvious enough.

For engineers' use it would be, of course, far less accurate than the stadia; but it might prove of value on certain rough reconnoissance work, where the use of tripod instruments is impossible, and rough measurements of distances are all that is required.

The instrument as delivered is said to be carefully adjusted to the scales used; but directions accompany it for testing and adjusting the glasses. It may be added that the instrument appears to be of excellent workmanship, and is exceedingly compact and light, weighing only about a pound. It is sold in the United States by Queen & Co., 1010 Chestnut St., Philadelphia, and we are indebted to the courtesy of that firm for the opportunity of examining the instrument, and preparing the above description.

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The rejection of the bids for the sewerage and paving of Havana, Cuba, gives American contractors yet another opportunity to secure this important piece of work. On the face of it, it seems strange enough that only one bid was received. That bid was submitted by the firm of Dady, Onderdonk & McClennan, of New York, and amounted to $13,855,575 for both sewers and paving. The City Engineer, Lieut. Wm. T. Barden, says that the Engineers' Department estimated the actual cost of paving at $4,561,495, including maintenance for five years; and to this they added 25% for contractor's profit and 40% for contingencies, a total of $6,386,093. The actual cost of the sewerage was placed at $3,516,888; to this was added 25% profit and 50% for contingencies, making $5,275,333. The two items make a grand total of $11,661,080, as contrasted with the bid of $13,855,575. With such liberal estimates as the engineers appear to have made, it is no wonder that the bids were rejected as too high, particularly in view of the fact that only one bid was received. The publication of the itemized bid, together with the engineers' estimate, also itemized, in our Construction News columns, will afford those interested an opportunity to judge of their chances on the new call for bids. We see but one reason, if ample time for the preparation of estimates is given, why there should not be active competition for this great piece of work, which has rarely been equalled in the history of municipal undertakings. That reason is the very magnitude of the contract, coupled with the requirements of a $500,000 deposit from each bidder and a further deposit of a second half million dollars from the successful contractor. If there is no legal obstacle in the way of a change of plan, might it not be worth while to try advertising the work in sections?

The unwarranted interpretation put upon certain decisions of the United States Supreme Court in the matter of assessments for street improvements has been pointed out by the same court in two recent decisions. Ever since the case of Norwood vs. Baker was reported (172 U. S. 269) it has been held in some quarters that all assessments for benefits made on the front foot basis are unconstitutional. In two recent decisions, involving Kansas City, Mo., and Tonawanda, N. Y., the United States Supreme Court clearly states that the Norwood case was a peculiarly gross instance of virtual confiscation of property, aimed against one property-owner. It also makes clear that the validity of the front foot basis of assessment, like that of any other system, depends wholly upon whether or not the benefit conferred equals the assessment. It is interesting to note that the Norwood decision attracted comparatively little attention in the East, but that in the North Central States, and even as far south as Missouri, it raised a great commotion. It appears, also, that in some of the Western States assessments were made in a very arbitrary manner, with little or no regard for questions of benefit or equity. The outcome of the whole matter has been to bring out more clearly and emphatically than ever before the fundamental principles governing this mode of meeting the cost of street improvements. The agitation will be a gain to the East as well as the West, for while the forms have been observed pretty generally in the East, close scrutiny of some assessments would show that little more had been done. In other words, there is often a tendency to be slipshod in apportioning assessments; to rely upon apparent rather than actual uniform treatment of the various property-owners; and to avoid all opportunity for trouble by keeping as quiet as possible about the detailed methods employed in arriving at individual assessments.

THE VENTILATION OF PASSENGER CARS.

"That some progress is being made in the automatic regulation of temperature in and the improved ventilation of passengers cars" is the conclusion of the American Railway Association's Committee on Safety Appliances, which has recently collected information upon this question incidental to a general investigation of the progress being made in the use of safety appliances by American railways. The evidence presented indicates, on its face at least, that the committee has not been under-sanguine in formulating its conclusion. Of the 80 railways which reply to the committee's general circular of inquiry only 17 answer the question regarding passenger car ventilation; of these only two appear to have actually in use anything different from the time-honored clere-story transom and open window method, and of these again only one words its reply as if it felt that a measurable and permanent advance were recorded by its new improvement. Viewed superficially, this statement of conditions might seem to indicate that railway operating officials had given scant thought to car ventilation and still less attention to its practical attainment. This would, however, be an unfair conclusion. As a matter of fact, railway officials have given the matter much thought in the past, and are still giving it thought, but they have discovered what the general public is to a large extent unaware of, namely, that the problem is one of extreme intricacy and cannot be solved off-hand, however desirable an immediate solution may seem to be. It is worth while, perhaps, to point out this fact in more detail, even at the risk of rethreshing old straw in doing it.

The usual method of ventilating passenger cars consists in taking in fresh air through open sash in the end doors and through movable transoms in the sides of the clere-story and in allowing the foul air to escape through similar or the same openings. This system is obviously far from perfect theoretically, and it is certainly not satisfactory in its practical working. Tests have shown, if tests were needed to demonstrate that which every person's senses have often informed him, that the air of passenger cars, and particularly of sleeping cars, is often more impure than is conducive to comfort, not to mention healthfulness. In a considerable measure doubtless this faulty condition is due to lack of intelligence and system in employing the means available for ventilation, but the fact remains that for one reason or another passenger cars as they are ordinarily ventilated are seldom well ventilated. How is this condition to be remedied? This is the question which confronts the motive power official. It is a simple enough matter to say that good ventilation merely requires that a certain number of cubic feet of vitiated air shall be expelled from the car each minute and that a corresponding volume of pure air should be taken in to replace it; but it is quite another matter to devise an efficient and satisfactory means of doing this under the conditions which ordinarily exist in a railway car in transit. These conditions are graphically expressed in the following paragraph from the report of the Massachusetts State Railroad Commission on car ventilation, which was published in 1896:

> The problem of car ventilation involves conditions which are peculiarly variable and complex. A car is sometimes in motion and sometimes at rest; sometimes it runs slowly, sometimes rapidly; sometimes it moves with the wind, sometimes against the wind, and sometimes at right angles to the wind; its doors and windows are sometimes open and sometimes shut. The passengers in a car, moreover, differ widely in temperament, health and habits of life, in sensitiveness to heat and cold and to drafts of air. Some are young and robust, others are aged or feeble. Some desire a high temperature, some prefer a low temperature; some want to sit by an open window, others cannot endure a current of air without taking cold; some are warmly clad, or retain in the car their overcoats and furs; others are thinly clad, or remove their outer garments as they would in a sitting-room. At stations, the doors of a car are necessarily thrown open, sometimes for several minutes; and they are also opened from time to time while the car is in motion, for the passage of the conductor and brakeman through the train.

It is evident to everyone that ventilation under such circumstances is a far more difficult problem than the ventilation of a building. The very first factor, namely, the amount of pure air required to preserve wholesome conditions in a passenger car, has not been generally established. The Pennsylvania R. R. in its new system of ventilation has decided upon 60,000 cu. ft. of air per car per hour, or 1,000 cu. ft. of air per passenger per hour. We shall not attempt to discuss here whether this is a reasonable figure or not, but it does not conform to the ordinarily accepted figure. Are we to accept the figure of 1,000 cu. ft. per person per hour adopted by the Pennsylvania R. R. or the figure of 3,000 cu. ft. per person per hour which is the one ordinarily adopted in building ventilation, or are we to expect some intermediate figure? It may be urged, perhaps, that a system based upon either of these figures would be preferable to that now in vogue, and there is doubtless much force in this contention. The point which we desire to make here is that with our present knowledge there is a wide opportunity for difference of opinion as to the amount of air required to ventilate properly a passenger car, and that both the mechanical and the financial difficulties of the problem hinge to a very large extent upon this point. A brief consideration of the conditions will show why this is so.

The air furnished to the car must at all times be free from dust, cinders and smoke, and in the winter it must be warmed. The larger the volume of air which is required to free from dust and to heat to a comfortable temperature, the more difficult and the more expensive is the task of doing it. A supply of 3,000 cu. ft. of air per person per hour means a total supply of 180,000 cu. ft. of air per hour to the whole car, or that the ordinary passenger car must be emptied of foul air and filled with fresh air about 45 times every hour. To provide for the continuous movement of such a volume of air without dust and without draft, and to ensure that it is comfortably warm at the moment of admission, is a problem of considerably greater magnitude than is encountered in any but the most difficult conditions of building ventilation.

Enough has been said, we think, respecting the general problem of passenger car ventilation to indicate its intricacy. In conclusion, it is interesting to note the lines of development which have been followed by the two railways whose answers to the American Railway Association's circular mention a change from the ordinary methods of car ventilation. The first of these railways is the Chicago & Northwestern, whose reply to the Association's circular is as follows:

FIG. 1. U. S. SPUR-DIKE B, MATTRESS AND ANCHORS, ON RIVER FRONT, 3rd DISTRICT, NEW ORLEANS, 1898. TO THE EXTREME RIGHT IS SEEN PART OF THE MATTRESS LAID IN 1888.

FIG. 2. U. S. SPUR-DIKE C, AND MATTRESSES, 3rd DISTRICT, NEW ORLEANS; AS FOUND IN 1898.

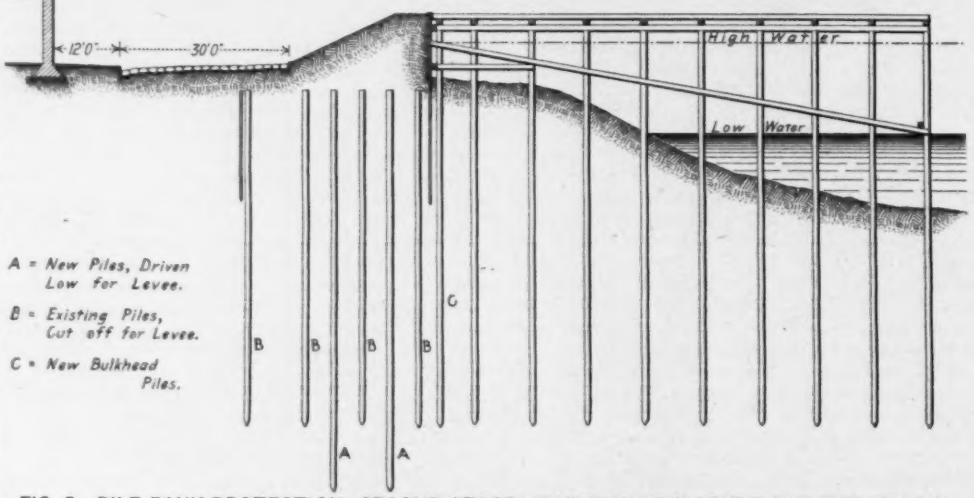

FIG. 8. PILE BANK PROTECTION; SECOND STAGE; SHOWING NEW LEVEE AND THE REOPENING OF THE RIVER STREET.

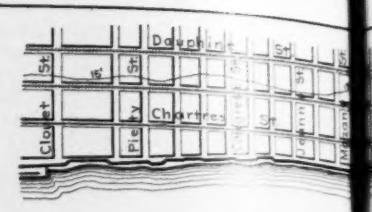

FIG. 4. THIRD DISTRICT, RIVER FRONT [illegible] NEW ORLEANS, SHOWING LOCATION OF THE BOX-LEVEES AND THE IRREGULARITY OF

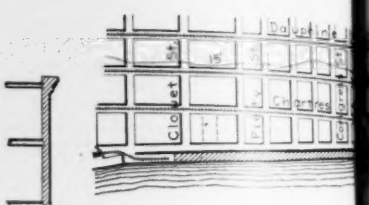

FIG. 5. THIRD DISTRICT, RIVER FRONT OF NEW ORLEANS, SHOWING LOCATION OF THE PILE BANK PROTECTION AND THE PRACTICALLY REGULARITY OF

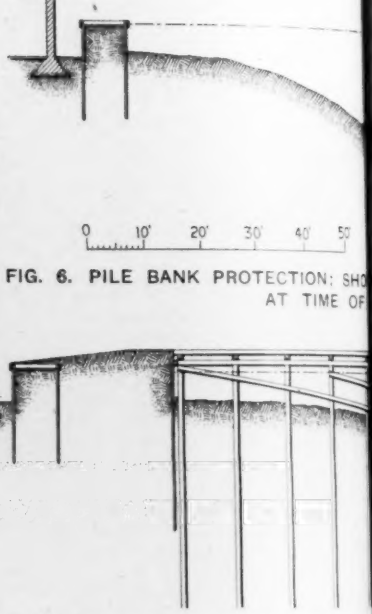

FIG. 6. PILE BANK PROTECTION; SHOWING MUD-BANK AT TIME OF COMMENCING

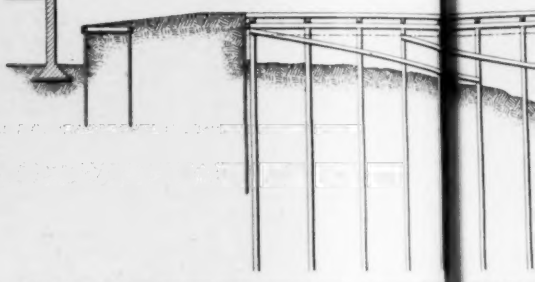

FIG. 7. PILE BANK PROTECTION; FIRST STAGE OF THE I

PROTECTION OF CI
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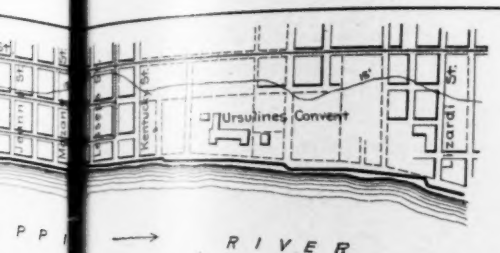
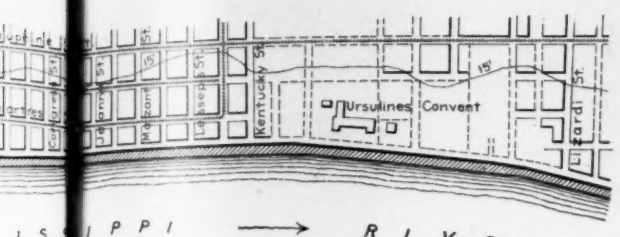
THE MISSISSIPPI R

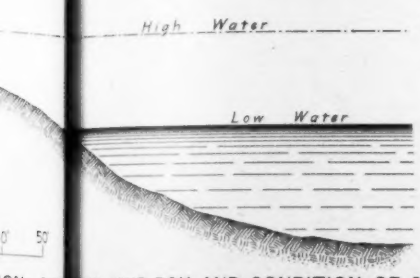
L. W. Brown, M. Am. S
Engineer in Char
New Orleans Levee B



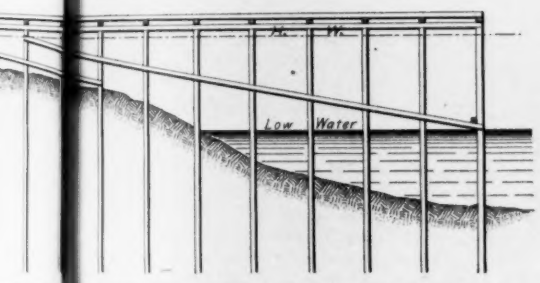
NEW ORLEANS IN 1897; SHOWING THE LOCA-
ULARITY OF THE WATER FRONT AT THAT TIME.



FRONT OF NEW ORLEANS IN 1900; SHOWING THE
ROTECTION WORK; REDUCING THE CONTOUR TO A
CALLY REGULAR CURVE.



ON: SHOWING MUD-BOX AND CONDITION OF SHORE
TIME OF COMMENCING WORK.



RST STAGE OF THE IMPROVEMENT; AS EXISTING IN 1900.

TECTION OF CITIES
HE ENCROACHMENT OF
MISSISSIPPI RIVER.

rown, M. Am. Soc. C. E.
ngineer in Charge
Orleans Levee Board.

FIG. 3. VIEW OF CAVING BANK AT GOULDSBORO, LA.
THE U. S. GOVERNMENT LAID A MATTRESS AT THIS POINT IN 1896; BUT BOTH LEVEE AND MATTRESS CAVED INTO THE RIVER IN 1898.

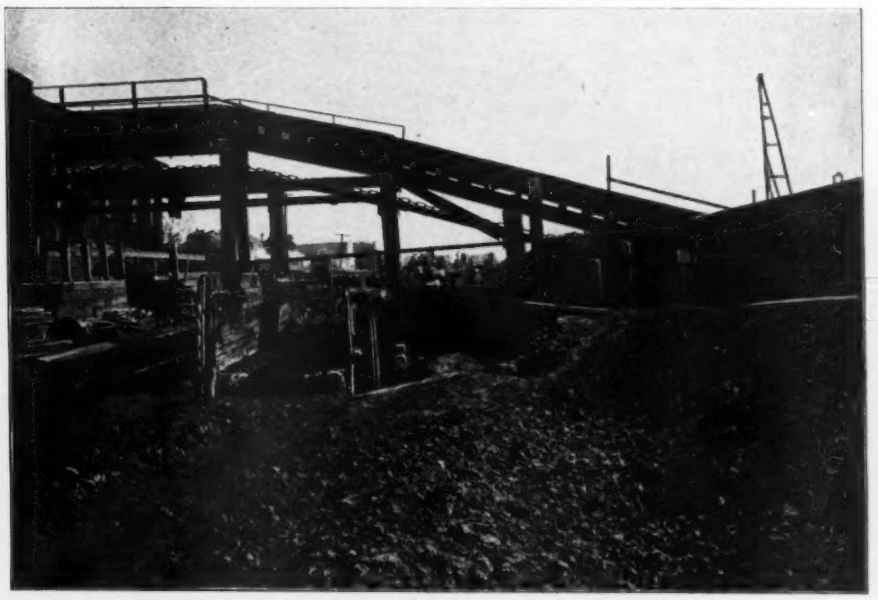


FIG. 10. THIRD DISTRICT LEVEE WORK; SHOWING BAD CONDITION OF THE OLD BOX-LEVEE.

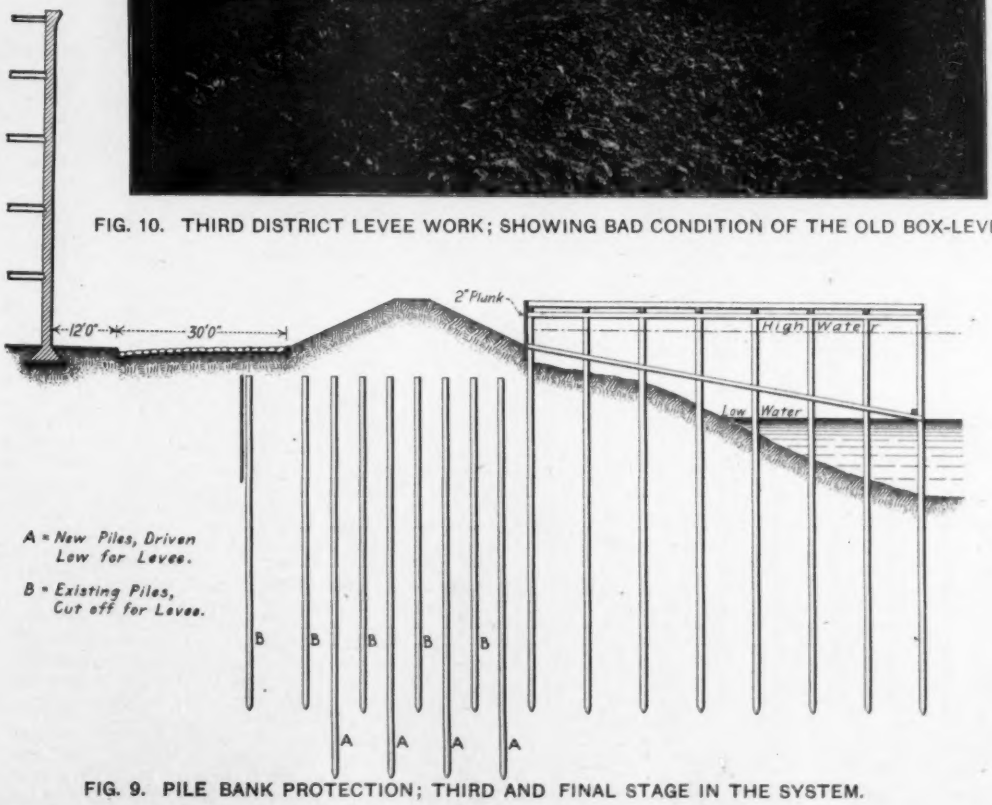


FIG. 9. PILE BANK PROTECTION; THIRD AND FINAL STAGE IN THE SYSTEM.

We have a system of ventilation for all day coaches consisting of an air-jack located at diagonal corners of the car directly inside the end; the pipe leading from this jack downward and discharging a quantity of air into the body of the steam radiator, where it is heated before it is discharged into the car; the discharge from the radiator is near the floor of the car; the jack at the rear end of the car acts as a vacuum and thus insures circulation. We have been unable to do anything as regards any special ventilation for special car equipment.

The second reply of significance comes from the Pennsylvania R. R., and is as follows:

After experimenting, a new system of heating and ventilating passenger cars has been developed. Briefly described, this new system is as follows: Air is taken in at diagonally opposite corners of the car through a hood or intake, allowing this air to pass down through a conduit underneath the floor of the car into the space bounded by the outside sill, the first intermediate sill and the floor and false bottom, this space extending the whole length of the car. From this space the air passes up through apertures in the floor into heater boxes situated alongside the "rock plank," where the air is warmed, and from each heater box the air passes out underneath every seat into the body of the car. Thence the air passes up through the globe ventilators in the deck to the outside air.

From an official statement of the work of the Pennsylvania R. R., published in the June number of the "American Engineer," we take the following statements concerning the operation of the system which it describes in its answer to the Association's inquiry. The amount of air admitted to the car is 60,000 cu. ft. per hour and the heating system required to warm it embraces 250 sq. ft. of radiating surface. Careful tests with the outside temperature about 13° F. have shown that there is no difficulty in keeping the car warm with the ventilating system in full normal operation, and that the distribution of the air through the car is satisfactory. These results were secured with the car in motion. With the cars standing and without heat, there is little or no movement of the ventilating current. Dust and smoke suspended in the air are not excluded from the ventilating current, but there is no trouble from cinders of appreciable size. So far few data have been obtained of the extra steam consumption for heating purposes which the new ventilating system entails, but it is thought that it will cause no serious drain on the locomotive.

Altogether, as will be seen, there is unusually good reason for thinking that in the new car ventilating system of the Pennsylvania R. R. a system has been developed which promises much hope for improvement in this important line of effort in catering to the comfort of the traveling public. That the Pennsylvania R. R. has expended nearly six years in experiment and investigation to develop this system to its present state is not the least notable part of its work, since it indicates that the problem of car ventilation, even when attended with unusual resources of expert knowledge and money available for experiment, is one which is beset with great difficulty.

A RESERVOIR RETAINING WALL AT OTTUMA, IA., Ia., was displaced by earth pressure a few days ago, but it does not appear that any water escaped. The reservoir was a small one, owned by the company that supplies the city with water.

LETTERS TO THE EDITOR.

Diagram Showing the Weights of Highway Truss Bridges.

Sir: The accompanying diagrams give the weight of highway truss bridges, both with and without sidewalks, for spans from 30 to 150 ft. These weights include steel only, without joists. The roadways range from 14 to 20 ft. in width and the walks are 5 and 6 ft. wide. The trusses are in all cases riveted, and those for through bridges have double systems, with inclined end posts and verticals at the first panel point only. Generally the top chords and end posts are of the usual box form, with two web plates and a cover, all connected by four angles. In the case, however, of bridges without walks and with spans from 30 to 55 ft. alternate weights are given for trusses having top chords made of two angles placed back to back. These are somewhat lighter than the box chords, chiefly on account of the saving in gusset plates and lattice bars. The number of panels is clearly shown on the diagrams. These bridges have wood joists and a floor composed of two layers of plank. The live load assumed is 80 lbs. per sq. ft. for trusses and 100 lbs. per sq. ft. for stringers and floor beams, or a 6-ton wagon in any position. The unit stresses for the main members are tension 15,000 lbs. per sq. in., and compression 12,000 lbs. per sq. in., reduced by Gordon's formulas; and for laterals the units are tension 18,000 lbs. per sq. in., and compression 15,000 lbs. per sq. in. The weights in all cases include bracing and shoe plates. These weights are conveniently expressed by the following formulas, where W is the weight of steel per square foot of floor, including both roads and walks:

$$\text{Truss bridges with sidewalks, } W = 2.8 + \frac{\text{span}}{11.3}.$$

$$\text{Truss bridges, without sidewalks, } W = 5 + \frac{\text{span}}{9.5}.$$

Very truly,
H. G. Tyrrell, Civil Engineer.
178 Devonshire St., Boston, Mass.

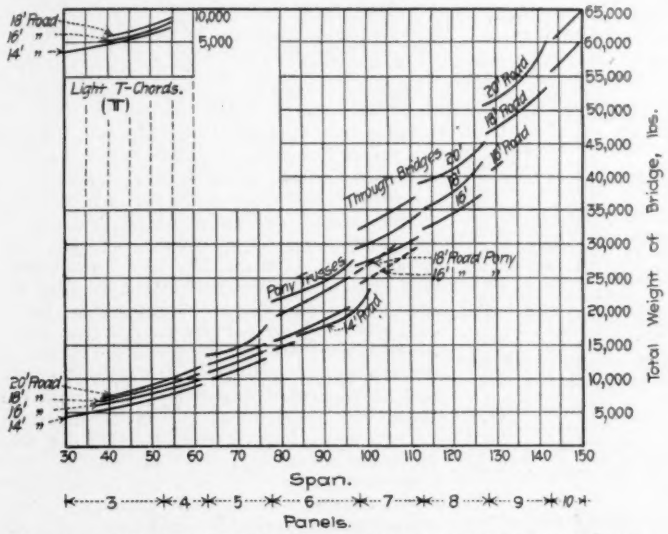


FIG. 1. DIAGRAM SHOWING WEIGHTS OF STANDARD HIGHWAY TRUSS BRIDGES WITHOUT SIDEWALKS.

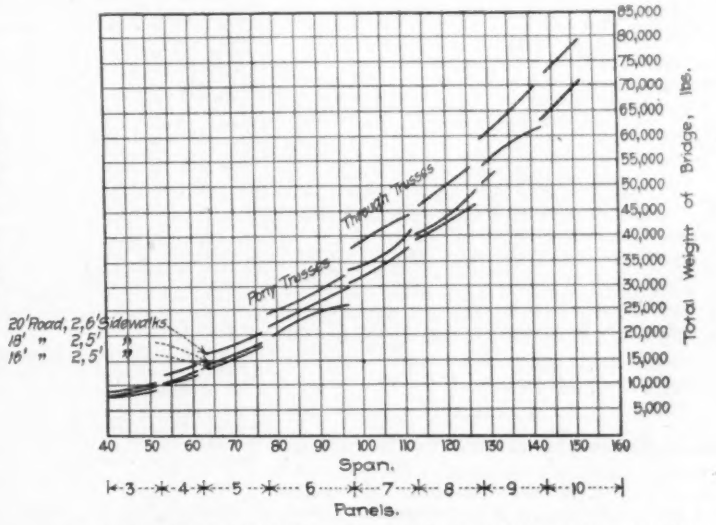


FIG. 2. DIAGRAM SHOWING WEIGHTS OF STANDARD HIGHWAY TRUSS BRIDGES WITH SIDEWALKS.

The Cost of Haulage by Horses and the Saving Effected by Road Improvement.

Sir: Since writing the letter that appeared in your issue of May 9, I have received the pamphlet upon which Prof. Baker bases his statement that the value of a team and driver is only $2.13 per day, and, as I had surmised, I find therein evidence that no sufficient allowance was made for lost time; for it is assumed that $1.50 per horse per month for pasturage during five idle months in the year is a fair allowance. Such an assumption is absurdly low, and in fact I find in Prof. Baker's own words a direct contradiction thereto. He says in the September, 1892, issue of "Good Roads," in reference to certain investigations: "The cost of feeding a team was found to be 40 cts. per day when working and 20 cts. when not working." In another place Prof. Baker adds: "The average load is almost exactly one ton. The average cost per load then becomes 25 cts. per ton mile," from which it would appear that Prof. Baker's earlier investigations and conclusions differ decidedly from his later ones.

But the point upon which I desire to lay particular stress is that in estimating the cost of team time he has not made sufficient allowance for lost time, an error by no means uncommon among engineers and others who have kept no actual account of the items of cost of work.

Referring to Prof. Baker's statement that I am far too low in allowing $320 per annum per mile for macadam road maintenance and interest, would state that his assumption that the $9,000 a mile roads of Massachusetts are fair examples is scarcely justified. In New Jersey the average cost of macadam roads has been about $4,500 a mile and New Jersey is a much fairer example of a state that has expended its money judiciously. Furthermore, I would point out that 3½% not 5% is the ruling rate of interest in these states where the security is good; so that where Prof. Baker's interest charges on a mile of road are $450 a year, the actual charge would be less than $175.

The average annual cost of maintenance in Massachusetts has been $105 per mile. While I believe this is ample where the roads are economically administered, it will be seen that there still remains quite a margin before the $320 is reached.

Prof. Baker says $320 is at least six times too high an estimate of the annual saving per mile due to macadam road construction. It should be remembered that in reaching the estimate of $320 a mile, I assumed a haul of only three miles, whereas the U. S. Dept. of Agriculture has found six miles to be the average haul in eastern states. I am aware that Prof. Baker questions the accuracy of the department's statistics, but whether they are exactly right or not makes little difference; for if six miles is not the present average haul over earth roads, it certainly will become as much as this or more, wherever macadam roads are built. With a six-mile average haul, an estimated saving is doubled, becoming $640 per mile per annum; and it must be remembered that this includes no allowance for saving due to the possession of a road that is equally good in all kinds of weather, an item which in itself is annually worth several hundred dollars per mile of road in a district where fruits, and "truck" are raised.

In his Fifth Annual Report on the roads of New Jersey, Mr. Budd gives an actual instance showing that since the building of macadam roads in Gloucester County the saving effected in hauling is $10 on each of the wagon loads now hauled, and adds that there are not less than 800 loaded teams daily going across a certain ferry, where formerly there were 100 to 150. Including the saving on commissions he estimates that upon each of the huge loads now hauled there is a total saving of $16. Of course this great saving is exceptional, but I have endeavored to be moderate in my estimates of saving, and when Prof. Baker says I am just six times too high, I turn to Mr. Budd, whose data would indicate that I am about six times too low, and feel that perhaps I have not been so extravagant in my claims after all.

Yours truly, H. P. Gillette.
791 Powers Block, Rochester, N. Y.

The Eccentricity of Object Glasses in Telescopes.

Sir: In your current volume, p. 377, Mr. S. P. Baird makes the statement, on the authority of several makers of scientific instruments, that a very large percentage of the object glasses manufactured are eccentric; "that is, it is impossible to so adjust the instrument that the intersection of the cross wires will cut points in a line, which is straight both horizontally and vertically."

The error is said to be thrown into the horizontal hair for transits; so that if a transit telescope was sighted, say along a level line at any point, then as the object glass is moved, the successive lines of sight through the horizontal hair, would not all be horizontal but would form a sheaf of planes.

A test of a Fauth & Co. transit for eccentricity of the object glass, based upon the "peg adjustment" method, is briefly indicated below, using 100-ft. stations, pegs were

put in at stations 0, 0 + 50, 1, 2, 3, 4, 5 and 6 and their elevations found by setting up the level in turn, midway between two adjacent pegs and sighting on them. By this line of levels, the true elevations of all the pegs were found, the curvature of the earth being considered. These elevations are given in column two of the following table:

Station.	Elevs. by level.	Elevs. by transit.	Diff.
0	10.000	10.000	
+ 50	9.204	9.205	+ 0.001
1	8.742	8.739	— 0.003
2	7.960	7.961	+ 0.001
3	6.220	6.225	+ 0.005
4	6.125	6.121	— 0.004
5	7.809	7.808	— 0.001
6	9.712	9.712	

Station 6 is thus 0.288 ft. below Sta. 0. The transit was now set up at Sta. 0 and the height of telescope axis above peg at 0, found to be 4.955. A target was now set on the road at the reading

$$0.288 + 4.955 + 0.007 = 5.250$$

(the 0.007 being the correction for curvature) and the rod held at Sta. 6. The transit telescope was next made to bisect this fixed target, so that it now sighted along a true horizontal line at Sta. 0. The object glass alone was moved to get the rod readings on stations 5, 4, 3, etc., in turn; but just before taking a sight, the fixed target at Sta. 0 was sighted at to correct any possible movement of the telescope. The correction for curvature was now subtracted from each rod reading in turn to get finally the elevations in column 3. The amount and character of the differences in the fourth column, do not reveal any eccentricity that is appreciable; in fact, the + and — errors look very much as if they were personal and instrumental errors solely.

It is plain that this transit can be used for ordinary leveling or in establishing grades by "the transit method." Let me, respectfully, offer to the profession, the latter designation (which is perfectly distinctive) for the awkward expression, "plunging the grade," which some have used. I have heard this method spoken of for 30 years past, but never knew it used before. It saves half the time in laying grades and is as accurate as the level method when no eccentricity of the object glass exists. The last can only be ascertained by an actual test of the instrument. Wm. Cain.

University of North Carolina,
Chapel Hill, N. C., May 31, 1901.

The Municipal Water and Light Plant at Brookhaven, Miss.

Sir: The accompanying statement from the city clerk of Brookhoven, Miss., relative to operating expenses as compared with revenue derived from the municipal water and light plant at that place will possibly be of some interest to readers of Engineering News, inasmuch as it bears directly upon the muchly-argued question of municipal ownership of public utilities.

The plant in question was constructed under the supervision of the writer as resident engineer during the fall and winter of 1897-8. Mr. Joe C. Guild, of Chattanooga, Tenn., was general contractor, and Sanders & Porter, of Louisville, Ky., were designing engineers. Briefly, the plant, as originally built, consisted of the following:

Brick power house, containing two 750,000-gallon compound Worthington pumps; air compressor for pumping wells; 45-light General Electric arc machine for street lighting; 1,000-light General Electric alternator for incandescent service; 120-HP. Filer & Stowell Corliss engine, driving both dynamos from a double-crowned pulley; two 80-HP. boilers; and the necessary switchboard and apparatus, feed-pump, heater and air chamber for compressor. The building contained a store-room and office, and ample space was provided for additional units of power and machinery, some of which, it will be noted, have already been supplied.

Water was obtained by sinking two 6-in. wells, provided with Cook strainers, into the water-bearing sand located 160 ft. below the surface. The Bacon air lift system was used to raise the water into a 90,000-gallon brick reservoir, built just outside the power house. The air lift was also successfully applied to the reservoir for clearing its bottom of accumulations of sand and silt without in the least interfering with the continuous operation of the pumps.

On the pipe line, also near the power house, was located one of the Chicago Bridge & Iron Co.'s hemispherical bottom steel tanks of 90,000 gallons capacity, placed on a steel tower 120 ft. high. The pipe system consisted of about 3½ miles of 10 to 4-in. pipe, with 52 double-nozzle Corey fire hydrants. Forty-three 2,000-c.p. arc lamps were placed on the streets. Immediately after construction almost the entire output of the incandescent machine was consumed by the demand for business and residence lighting. To encourage this the city for a limited time furnished free open cleat wiring to all desiring the service. The charge for incandescent lighting was below the average, being arranged on a sliding scale, but averaging only about 33⅓ cts. for 16-c.p. lamps. Water for residence use, sprinkling and the like also became popular from the first, and added materially to the revenue, etc.

The cost of the entire system, as originally constructed, was about $55,000. This was met by an issue of 5% 20-year bonds, sold slightly above par. Since construction additions to the extent of $15,000 have been made, making total cost to date about $70,000.

For fuel during the first month or two coal was used, but experiments with the refuse supplied by one of the neighboring sawmills proved very successful, and led to the use of this material altogether. This no doubt largely accounts for the comparatively low cost of fuel as given in the statement of expenses.

The statement of the city clerk, given under date May 22, 1901, is as follows:

OPERATING EXPENSES AND REVENUES OF CITY WATER AND LIGHT PLANT.

Expenses:

Superintendent's salary, per month	$100
Two engineers, at $50 per month	100
Two firemen, at $40 per month	80
Fuel, per month	265
Oil and waste, per month	20
Repairs, etc., per month	25
Total operating expenses, per month	$590
Total revenue from light and water, per month	850
Excess of revenue over expenses	$260

There are in use at present 1,850 incandescent lamps, averaging about 33⅓ cts. per lamp, and 217 water taps, at an average price of $1.25 each per month.

There are in use at this time, for which the city makes no charge, or from which the plant gets no credit, 43 2,000-c.p. arc lamps and 52 fire hydrants. The city considers these as free, inasmuch as the plant overpays operating expenses.

Since the installation of the plant, about three years ago, there has been added one 120-K-W. alternator and one 65-light improved arc generator, as well as a 90-HP. high-speed engine. We have also sunk a new 8-in. well and made improvements altogether amounting to about $15,000. Within 30 days' time we will add to the plant a 150-HP. boiler and a large air compressor capable of running all three wells at one time.

The plant is on a continual increase, and as the town grows the popularity of the system increases proportionally. The water supply, as is well known, is absolutely pure, and has never shown the least signs of exhaustion. As soon as the new compressor is installed we will enter into a contract with the railroad company for supplying their water tank here at $100 per month, thereby making considerable increase in the city revenue.

The plant is working very nicely indeed, and has never given us the slightest trouble in any way. This is the best proof of good workmanship and construction.

(Signed.) W. H. Seavey, City Clerk.

If we allow fair average prices on arc lights and fire hydrants, we get the total revenue of the plant in excess of operating expenses as follows:

43 arc lights, at $75 per year	$3,225
52 fire hydrants, at $50 per year	2,600
Excess of actual cost, revenue over operating expenses	3,120
Total	$8,945

Or about 12¾% on the $70,000 invested; and this will no doubt be materially raised when the $1,200 additional contract is made with the railroad company. As it is, the city is getting its own water and lights free, and in addition taking in almost enough revenue to pay the interest on its bonds. Granbery Jackson, C. E.

Mount Pleasant, Tenn., May 28, 1901.

(We publish the above as an interesting general description of a small combined water and lighting plant which appears to be giving excellent satisfaction, financially and otherwise, but we do not think the figures sufficiently full to prove anything as to the relative economy of municipal and private ownership. The expense account seems to be lacking in a number of items that inevitably arise in the operation of both water and lighting plants, such as carbons for the arc lamps, to name only one.—Ed.)

SWEDISH METHODS OF PREPARING PEAT-FUEL.

Any information relating to the economial production of peat-fuel is of interest in both the United States and in Canada; and the following matter relating to Swedish methods of peat-fuel treatment is taken from an article in "The Engineer," of London, evidently prepared from the circulars of the manufacturers of the machinery.

In the Fredriksson system of handling peat, the inventor uses processes more or less familiar in this country. He raises the cut peat from the bog by a belt-elevator; stacks it in the air along lines of portable railway track; removes it to sheds; gives it a final drying by artificial heat and then compresses it into briquettes. He does not fully detail the drying process other than to say that the peat is carried continuously through a long rotating iron drum, with the steam generated by the moisture in the peat utilized in the drying process.

In the treatment of peat Nils Fredriksson assumes that the peat as taken from the bog contains 50% of moisture, and that this moisture has to be reduced to 15%; if reduced below 15%, he says that valuable combustibles will be lost. He says that a metric ton, or 2,204 lbs. of 15% moist peat contains 1,870 lbs. of really dry peat, and to produce this requires 3,740 lbs. of 50% moist peat, or 1,540 lbs. of water have to be evaporated. Assuming that 1 kilo. or 2.2 lbs. of peat will yield 3,000 heat units, he figures that 127 kilos. or 279 lbs. of peat must be burned to evaporate the quantity of water mentioned. A machine tears the peat into shreds before the drying by artificial heat is performed, and the briquetting is done by a special machine which compresses the near-dried peat into small blocks.

With a daily output of 20 tons of 15% moist briquettes, working the plant 300 days in the year, the annual output would be 6,000 tons, requiring 10,200 tons of raw peat containing 50% of moisture. The cost of gathering the peat in Sweden averages 54 cts. per ton; and the cost of manufacture is set down in detail as follows, translated into American money:

Gathering 10,200 tons of raw peat at 54 cts.	$5,5[illegible]
Drying 6,000 tons, at 300 kilos. of 15% peat per ton, or 1,800 tons of 50% peat, at 54 cts.	97[illegible]
For 50-HP. steam engine, 600 tons peat, at 54 cts.	32[illegible]
Six men, or 1,800 days' labor, at 81 cts.	1,45[illegible]
Foreman, per annum	54[illegible]
Cost of loading	49[illegible]
Extra expenditures	51[illegible]
Total cost of 6,000 tons of briquette, say $1.62 per ton	$9,71[illegible]

The cost of a plant of this capacity, not including cost of the peat-bog and the site for buildings is set down at $17,250 or 75,000 kroners, the Swedish kroner being taken at 27 cts. in U. S. money.

The system of treating peat for fuel, invented by Aleph Anrep, is considered the best, in both Sweden and Russia. The Anrep machines are made by the Munktell Mekanista, Verstag, Eskilshima, Sweden, The peculiar merit of these machines is that they convert the raw peat into rectangular rope-like masses, which are cut into convenient lengths and dried in the air, and are fit for fuel without going through a special briquette machine.

The Anrep apparatus consists, first, of an elevator for lifting the peat from the bog. This is made of a strong iron chute, 34½ ft. long, through which runs an endless cable, over two rollers at the ends, and carrying a wooden bottom made of slats. The cable carried the peat up the chute on these slats and returns on the under side of the chute. The peat thus lifted is dumped into a cutting machine which is made of cast iron in the hollow body. Inside this body are two parallel axles fitted with strong steel knives and compressing rollers and grooves, operated by gearing , rotected by a cast-iron rim. The 14 axe-shaped knives on the axles cut, scissor fashion, against an equal number rising from the bottom of the box. The peat thus thoroughly cut up and squeezed by the compressing rolls and grooves, is forced into a steel shaping outlet and passes onto boards running on rollers. The peat as it leaves this machine has a section of about 5 × 4½ ins.; this peat-rope is then cut into pieces about 13 ins. long, is loaded onto wagons and is taken to the drying field.

The cutting machine is strong enough to dispose of ordinary roots; but should a stone get into the cutter, the belt is thrown off and the machine stops. The knives make 235 revolutions per minute; and the thorough treatment of the raw peat in this machine causes the air-drying process to be much more rapid in effect; the reduction in volume is as much as one-eighth of the wet peat, and the resulting product is hard and compact.

In a late Russian competitive trial of peat-fuel machines, an Anrep machine, operated by 22 men, turned out 6,353 blocks of peat-fuel, ready for drying, per hour for two days. About 500 of the Anrep machines are now used in Russia, and the output from them varies from 35,000 to 70,000 pieces per day, of the dimensions given, according to the character of the bog. The air-dried peat from these machines contains about 20% of moisture. It is advised, however, that the bog should be well drained to its working depth, and ample and well-drained drying areas should be provided. The wagons for carrying the peat-blocks to the drying field are fitted with three wooden shelves, holding 100 blocks of peat. The total cost of an Anrep plant is $4,050, including elevator, peat-cutter, portable engine and transportation outfit.

THE DESIGN OF ARCH CULVERTS.

By Daniel B. Luten.*

The standard designs for arch culverts adopted by most American railways are fairly represented in the accompanying cut, Fig. 1. The arch consists of a semicircular vault, supported on side-walls or abutments about equal in height to the radius of the semicircle, so that the interior height of the structure from the bed of the stream to the

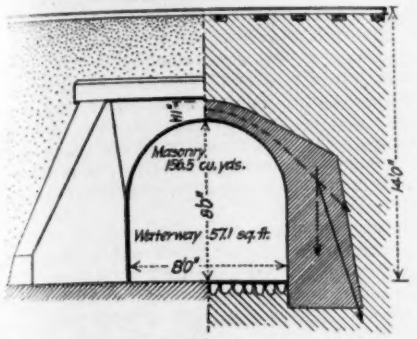


Fig. 1. A Common Design for Arch Culverts.

crown of the arch is approximately equal to the span.

Such a type of arch structure is simple and easy of construction, but it is not economical of material. Even when covered with an extremely heavy fill, stability and economy cannot be claimed for it in comparison with arched culverts of other forms. This is due to the unfortunate combination of the semicircular arch and vertical side-walls. The semicircle is ill-adapted to resist vertical loads, or moving loads, and hence is not suited to light fills; it is better adapted to an extremely heavy fill, which, however, endangers the vertical side-walls by the heavy lateral thrust.

Some advantages of segmental arch culverts over the more usual semicirucular type have been pointed out by Prof. I. O. Baker, in his text book on "Masonry Construction." With the increasing use of concrete and brick in railroad structures, a much more radical stand may be urged in favor of the arch of slight rise and long span. With this point in view, the design shown in Fig. 2 is submitted for purposes of comparison with Fig. 1. The area of waterway of the flat arch is slightly in excess of that of the semicircular arch. Yet the flat arch has the following additional advantages over the semicircular arch: It requires from 10 to 20% less material, with a saving in cost of from 15 to 25%. It will discharge from 50 to 90% more water in a given time. It is less liable to become obstructed by debris. It is better protected from the vibrations of traffic. It will suffer less from erosion in case of floods. It is adapted to shallower fills.

The arch shown in Fig. 1 has an area of waterway of 57.1 sq. ft.; from the bed of the stream to the base of the rail is 14 ft.; the cross-section consists of a semicircle of 8 ft. diameter, resting upon side-walls 4 ft. high. According to the standard designs of a well-known western road, the thickness required for the arch ring at the crown is 11 ins., leaving a depth of fill above the top of the arch of 5 ft. 1 in. The side-walls, or abutments, carried to a depth of 18 ins. below the bed of the stream, should have a base of 3 ft. 6 ins., and the thickness of abutment at the springing line of the arch would be 3 ft. The bed of the stream is paved with cobblestone grouted with cement mortar, with cross-walls and apron. End walls with coping and splayed wing-walls for a 1½ to 1 slope complete the design.

The design shown in Fig. 2 has an area of water-way of 58.3 sq. ft.; from bed of stream to base of rail is 14 ft, the same as in Fig. 1. The span is 16 ft, the height 4 ft. 6 ins.; the thickness at the crown is 16 ins., leaving a depth of fill above the arch of 8 ft. 2 ins. The form of the section is a three-centered curve resting upon vertical side-walls 6 ins. in height. The foundation walls carried to a depth of 18 ins. below the bed of the stream have a base of 6 ft.

*Los Angeles, Cal.

The amount of material required for the semicircular arch, for each foot of length, is 71 cu. ft. of masonry, and 8 sq. ft. of paving. For the flat arch, 64 cu. ft. of masonry is required per foot of length and 16 sq. ft. of paving. Fig. 2 has, however, been designed to be above criticism as to stability, and a reference to the equilibrium curve for steady load, drawn in dotted line, will show that the arch ring follows the curve closely, and that the resultant of the thrust and of the weight of the abutments, falls well within the base of the foundation wall. In the other arch, while the equilibrium curve does not deviate far from the semicircle, the resultant does not fall properly within the base of the abutment, thus depending upon the earth backing of the abutments to sustain a large part of the thrust.

The resolution of the forces shows that the masonry of the semicircular arch will be subjected to greater intensity of stress than will the masonry of the flat arch; also, that the maximum intensity of pressure upon the earth foundation and the earth backing of the abutments is greater in the semicircular arch. Moreover, the application of a concentrated moving load to the roadbed of Fig. 1 will cause a greater deviation of the equilibrium curve from the center of the arch ring than would occur in the flat arch, both because of the form of the arch ring and because of the lighter fill over the arch. The flat arch, having 50% more fill over the crown, will suffer less from the pounding and vibrations of swiftly-moving traffic. According to Trautwine's formulas, Fig. 1 has not sufficient thickness at the base of the abutments for safety; the thickness at the crown and at the springing agree with the dimensions by Trautwine. According to Baker's "Masonry," Fig. 2 has a greater thickness at the crown than is

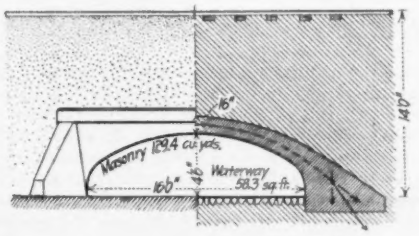


Fig. 2. Proposed Flat Arch Culvert.

necessary, and the abutments have a greater bearing area than is essential. The flat arch is, therefore, a better designed structure than the semicircular arch of Fig. 1, and is consequently less liable to accident or failure.

The low, flat arch will necessarily be of greater length than the other in order to meet the side slopes; but this is counteracted by the necessity for heavier wings and walls in the semicircular arch. Thus, for a 16-ft. roadway, the semicircular arch will require 156.4 cu. yds. of masonry, while for the same width of roadway the flat arch will require 129.5 cu. yds., a saving of 17½%. If concrete be used, an additional saving is made in the amount of lumber required for forms, as well as in the labor required for erection of the forms, and placing the concrete. The flat arch will, however, require more paving and excavation than the semicircular. The former requires 67 sq. yds. of paving, the latter 25, or, if the floor be made of concrete, 14½ cu. yds. for the former and 5 cu. yds. for the latter.

The most decided advantage in favor of the flat arch is the fact that for a given flood height it will discharge nearly twice the amount of water; for when the flat arch is flowing full with a depth of 4½ ft., the semicircular arch would be flowing but 62% full. For a height of flood that would completely fill the semicircular arch, the flat arch would be discharging under a head of 3½ ft. above the crown. The discharge area of the flat arch might in fact be reduced 30% with a proportional decrease in the cost of construction, and that arch would still be capable of safely disposing of heavier floods than the semicircular arch.

Because of the greater discharging capacity of the flat arch, Fig. 2, it would be able to dispose of a given amount of flood water with less velocity of current than the semicircular culvert, and would consequently be less endangered by erosion of the bed of the stream. The flat arch being of greater span, is, of course, less liable to obstruction by floating timbers and other debris.

It may be urged that the three-centered flat arch is not as handsome a structure as the semicircular arch. This is a point of small importance to the engineer designing small railway culverts for locations where passengers over the road will rarely obtain a view of the form of section. The appearance of an elliptical arch depends largely upon the amount of fill over the crown. With very light fill and long span it becomes a handsome form; with heavy fills, the semicircular form presents the better appearance. For important structures, therefore, the flat arch has an advantage in point of appearance.

For highway bridges the same reasoning applies; the ellipse is a handsome form if not covered by too heavy a fill; and in the case of small culverts beneath heavy fills, the advantages pointed out are too numerous to be outweighed by the matter of appearance.

The writer does not wish to be understood as advocating the elliptical form as the most economical of all possible types of arch culverts. An approximation to a catenary curve would be more nearly in equilibrium under the usual loading and would consequently require less material, but might be less economical when area of waterway combined with long span and slight rise are taken into account.

EXPERIMENTS ON SEPTIC TANK ACTION AT CORNELL UNIVERSITY.

By R. B. Williams, Jr.*

Since the introduction and use of septic tanks in connection with sewage purification, the "rest period" of the sewage, that is, the length of time required for the sewage to pass through the septic tanks, has in different plants varied from 1 hour to 36 hours, as the ideas of the engineer or the lack of area, funds or sewage have determined it. That temperature, concentration and freshness of the sewage affect the length of the most effective rest period, is quite generally known, but definite facts on the subject seem to be lacking. There are in this country, as well as abroad, several places where careful studies of these matters have been made for the existing local conditions. The Lawrence Experiment Station has made valuable, general studies of septic tank action, and Washington University is carrying on similar studies. These investigations and reports of the subsequent workings of the tanks are necessarily our guides in designing new works. A valuable improvement in the practical design of tanks has been made recently, known as the elastic tank. The tank for Holland, Mich., for example, is supplied with compartments and gates whereby the rest period can be readily adjusted

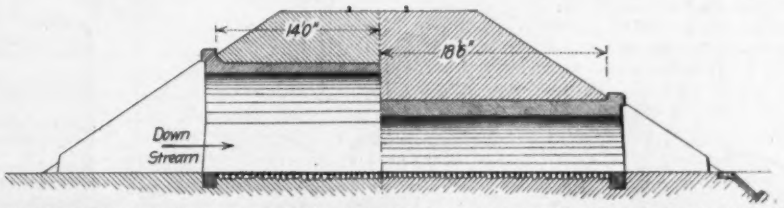


FIG. 3. HALF LONGITUDINAL SECTIONS AT CROWN OF SEMICIRCULAR ARCH AND OF FLAT ARCH.

*Of the graduating class in civil engineering, Cornell University, Ithaca, N. Y. (P. O. Box 355.)

within certain limits. This is a step in the right direction, but there is still great need for definite data on this important subject.

To further the knowledge along these lines, there have been started at Cornell University, Ithaca, N. Y., by the author of this article, five experimental septic tanks, with six filters in connection, designed by him especially for the study of the effects of the various rest periods. The five tanks

VIEW OF EXPERIMENTAL SEPTIC AND FILTER TANKS AT CORNELL UNIVERSITY.

are formed by partitions in one large tank and are similar in all respects, each containing approximately 120 cu. ft. of sewage. The inlet weirs are triangular, and movable up and down, thus facilitating any desired change in the rest period. The flow into the tanks and the outflow from them are slightly above midway between the bottom of the tanks and the floating scum, and so designed as to prevent any formation of currents within the liquid.

At present the positions of the inlet weirs are such as to supply flows that will require 2, 4, 8, 12 and 24 hours, respectively, to pass through the tanks, using a total daily flow of 20,000 gallons. Narrow weirs, carefully rated and read by piezometers, are placed at the outlets to determine the exact flow from each tank, thus giving data to check the flow over the inlet weirs, and the length of flow through the various tanks. Thermometers are placed in each tank for reading the temperature of the sewage therein.

The filters are each 1-5,000 acre in area, with 6 ins. of gravel and 36 ins. of coarse clean sand, known as No. 10 filter sand. Five of the filters receive the septic tank effluent intermittently, one from each tank, having four hours' flow and eight hours' rest, while the sixth, which is of the Scott-Moncrieff pattern, with six tiers of sand and gravel, with air spaces between, receives a continual flow from the third septic tank, the tank having at present an eight-hour rest period. A side investigation is thus possible in the comparison of the Scott-Moncrieff continual flow filter and the ordinary intermittent filter, which receives its supply from the same tank. For this experiment, the rate of flow on all the filters is 1,000,000 gallons per acre per day; that is 200 gallons per day on each filter. This flow may be very accurately regulated; the intermittent flow, both by weir adjustments and by changes in the electrical clock which opens and closes the valves leading to the filters; and the continual flow by weir adjustments only. By means of a series of narrow weirs, and broad waste weirs, forming what might be called a set of "differentiating weirs," the flow to the filters may be so accurately adjusted that any variation of head whatever, above the required amount for the filters, behind the first of the series of weirs, does not affect the volume of the final flow in the least, the waste weirs carrying off all excess. This was proved by experiment before the boxes were set in position. The excess sewage, excess flow through the tanks, together with the filter effluents, are carried to waste in a nearby creek, not afterwards used for drinking purposes.

Suitable arrangements are made for the taking of samples for chemical and bacterial analysis from the sewage in the inlet box, from the effluent of each septic tank, and from the effluent of each filter, making twelve samples in a set. One set of samples is now being analyzed daily, the author being assisted in the chemical analysis by Mr. H. C. Troy, State Chemist, stationed at Cornell University; and in the bacteriological analysis by Mr. O. F. Hunziker, graduate student in bacteriology. For the first month or so these analyses will show the growth of efficiency of the tanks and filters, while later they will be studied more closely for the problem in hand.

The sewage supply is obtained from the outfall force main, just after it has left the pumps in the pumping station, and is therefore free from all large masses of organic matter, such as would be caught on an inch roughing screen. The "separate system" of sewerage is employed in Ithaca, and no manufacturing wastes whatever enter the sewers.

The accompanying photograph shows the arrangement of the tanks, filters and connections. The raw sewage enters the tank at the rear, and passes down into the body of the tanks by pipes, not shown. The septic tank effluent passes up again through similar pipes to the outlet distributing boxes, containing the "differentiating weirs." These boxes are located between the tanks and the filters, which last are shown in the middle and at the front. Vertical 6-in. box troughs are placed at each end of the tanks, as wasteways for carrying off the excess sewage and effluent, and are not necessary to the experiment proper. The filters discharge into troughs, one of which is shown in the view. The arrangement of the Scott-Moncrieff filter is also plainly shown, and the arrangement of the differentiating weirs and the spider-shaped spattering apparatus for aerating and the spreading the septic tank effluents over the surface of the filters are brought out quite clearly.

The exceptionally favorable conditions at Ithaca, resulting from a combination of typical house sewage, entirely free from chemical impurities, ease of obtaining sewage without pumping, and the opportunity of continual supervision by the College of Civil Engineering, Cornell University, makes this a desirable place for the conduct of many future experiments in this line.

FILTERS FOR REMOVING OIL AND MUD FROM BOILER FEED-WATER.

One of the most important problems connected with a steam plant is the providing of pure water for the boilers. Sometimes, as when spring or well water is used, the chief difficulty is with matter held in solution, but in the case of river water or water from surface condensers the material to be removed is usually held in suspension. Sand or other sediment can be partially removed in settling

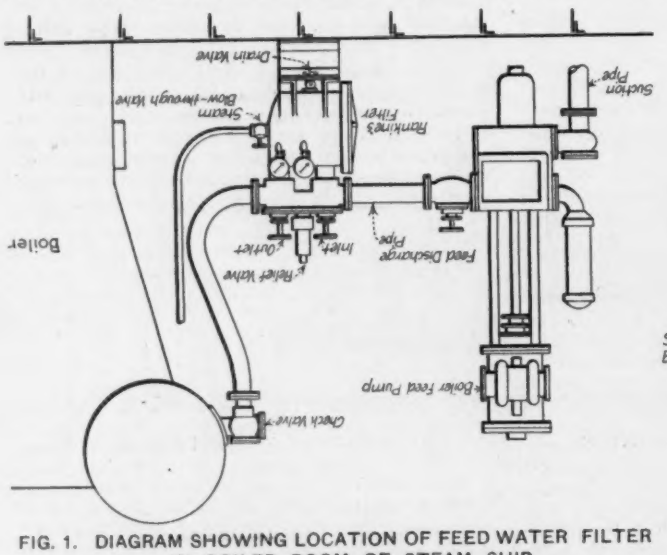


FIG. 1. DIAGRAM SHOWING LOCATION OF FEED WATER FILTER IN BOILER ROOM OF STEAM SHIP.

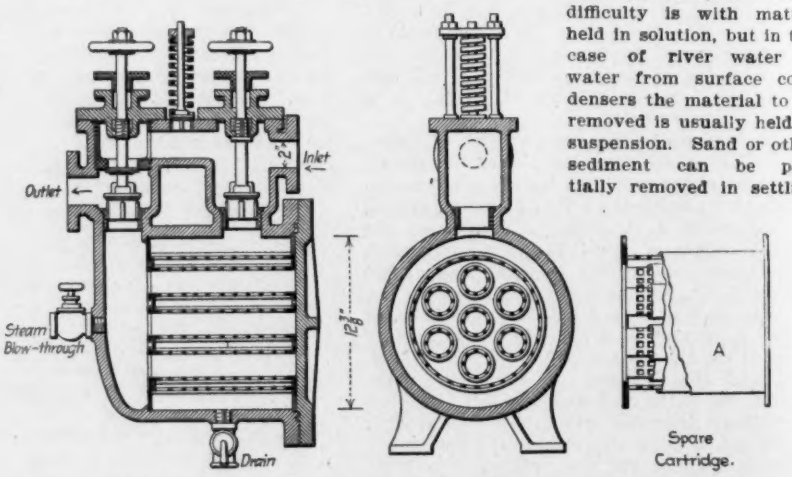


FIG. 3. DUPLEX FEED-WATER FILTER FOR USE WITH MUDDY WATER.

tanks and oil can be gotten rid of to some extent by steam separators or skimming devices, but the only effective and sure way of removing either is by straining or filtering. To accomplish this some device must be provided which is light and compact, in order that it may be installed in crowded boiler rooms or on board ship, and which is so constructed as to permit the easy removal and renewal of the filtering medium. One of the

best materials for filtering has been found to be a special grade of Turkish toweling.

The filter which we are about to describe has come into extended use abroad, both in land and marine boiler plants. It is in use on warships of the British, German, Russian and other navies and on many merchant vessels. It is being introduced in the United States by Mr. W. Blackburn Smith, of Perth Amboy, N. J., who has furnished us with blue prints from which the accompanying illustrations have been prepared.

In filtering muddy water a special apparatus is necessary. Since the mud quickly fouls the filtering material, it is essential to have the filter arranged with two chambers, one of which may be cleaned while the other is in operation. A filter specially designed for use with muddy water is shown in Fig. 3. As there seen, two filtering chambers are placed side by side, and are connected by a "Y" casting. The cartridge used for filtering muddy water consists of a large outer cylinder, covered with brass gauze and filtering cloth, and a small inner cylinder also covered with gauze. The space between is packed with some fibrous material.

In order to do away with the necessity of frequently removing the cartridges, a steam valve is fitted at the end of each chest, whereby steam is admitted in the reverse direction to the flow of the feed water. The drain valve being opened and the inlet and outlet valves closed, the mud is blown out through the drain valve.

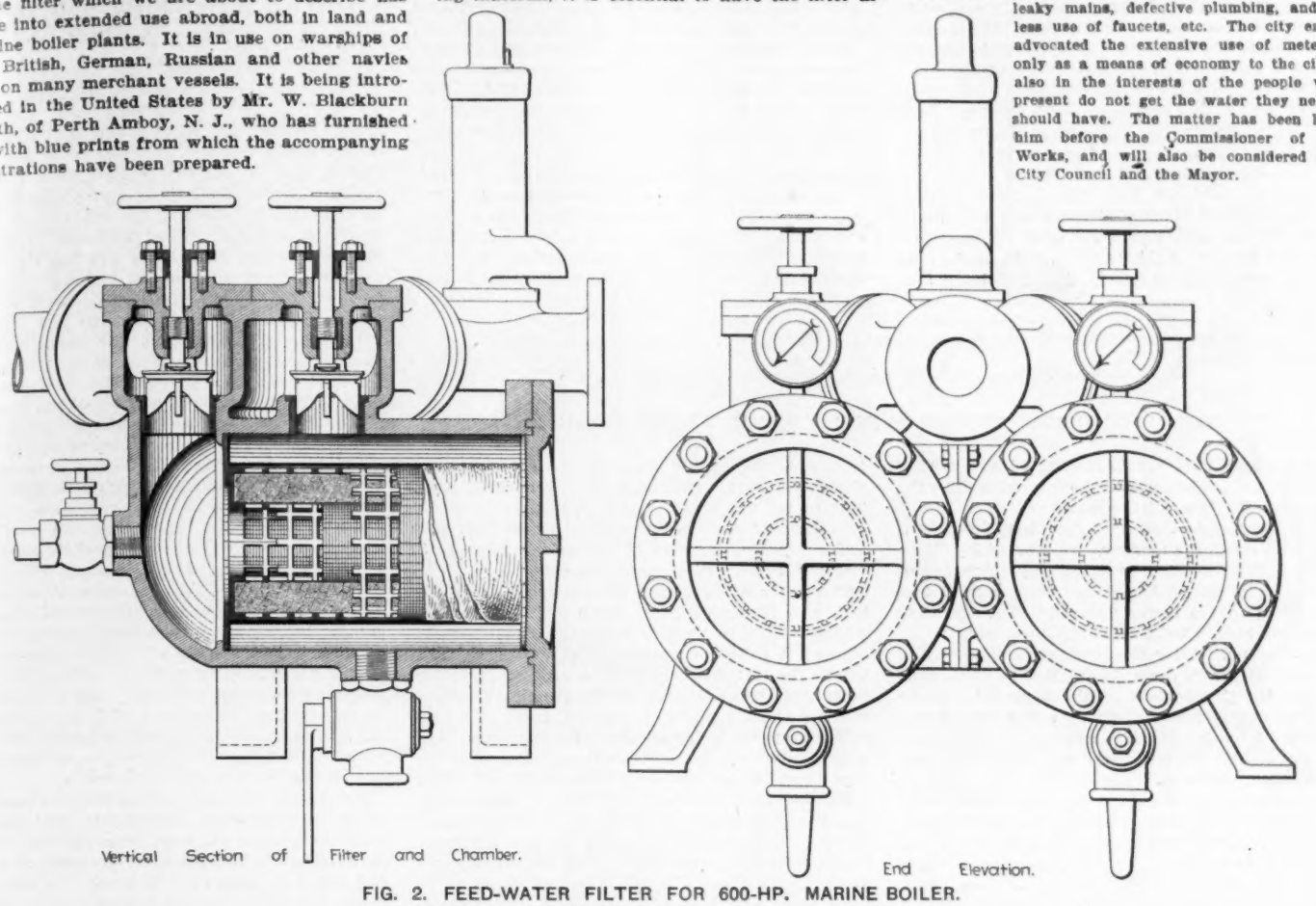


FIG. 2. FEED-WATER FILTER FOR 600-HP. MARINE BOILER.
Manufactured by W. Blackburn Smith, Perth Amboy, N. J.

The manner in which the filter may be installed in the boiler-room of a steamship is illustrated in Fig. 1. The filter is placed on the feed pipe between the pump and the boiler, so that any desired amount of pressure may be used to force the water through the filtering material. This pressure is limited by a relief valve which automatically by-passes the filter when the resistance to the flow of water through the latter becomes more than about 20% of the boiler pressure, and pressure gages may be provided which will show the drop in pressure through the filter at all times.

A filter now being constructed for the tug "Mabel," of the Hartford & New York Transportation Co., is illustrated in Fig. 2. The holder for the filtering material is shown at A removed from the case. It consists of an outer cylindrical frame and within this a number of smaller cylinders, all composed of bronze or other suitable metal. Both the large outer cylinders and the small inner cylinders are wrapped with Turkish toweling. As may be seen from the longitudinal section, the water passes first through the covering of the outer cylinders and then through the coverings of the smaller cylinders. This double filtration has been found necessary in order to completely remove all traces of cylinder oil from water from surface condensers.

Another arrangement for securing double filtration, adopted on large size filters for naval use, is to provide a central longitudinal partition in the filter shell and so connect the filtering "cartridges," as they are called, that the water passes first through the cartridges on one side of the partition and then through those on the opposite side. Where an especially high degree of purity is desired filters designed to give three successive filtrations have been constructed.

All filters are supplied with a spare set of filtering cartridges. The four cartridges are removed and replaced with clean ones as soon as the pressure through the filter reaches 15 or 20 lbs. per sq. in. The foul cloths are washed out in boiling water and can be used a great many times.

AN AIR TORPEDO, invented by Major Urigo, of the Swedish army, is noted by Consul General R. Guenther, of Frankfort. The propelling force is obtained from a gas generated by slow-burning material contained in the torpedo, with the pressure gradually increasing. The initial velocity is slight, thus bringing slight pressure upon the torpedo gun. A flight of 16,000 ft. has been recorded in the trials.

THE WATER SUPPLY OF CHICAGO was discussed by Mr. John Ericson, City Engineer, at a meeting of the Western Society of Engineers on June 5, in a paper supplementing one previously presented by Mr. J. H. Spengler, Assistant City Engineer. With a pumping capacity of 539,500,000 gallons, and a present daily pumpage of 360,000,000 gallons, or 177 gallons per capita, it might seem that further enlargement of the supply would not be necessary for a long while to come. As a matter of fact, certain sections of the city, particularly south of 87th St., are now complaining of a shortage in the supply, and Mr. Ericson has recommended new lines of mains from some of the new pumping stations, as well as the construction of a new intake tunnel from the 68th St. crib to a new pumping station near 103d St. and the Illinois Central R. R. The most important step, however, will be to take some means to check the present enormous waste. In February last, when the weather was above freezing (so that taps would not be left running) and no water was being used for sprinkling, the waste ranged as high as 76% of the total pumpage, which is a most extraordinary record. This waste is due in part to old and leaky mains, defective plumbing, and reckless use of faucets, etc. The city engineer advocated the extensive use of meters not only as a means of economy to the city, but also in the interests of the people who at present do not get the water they need and should have. The matter has been laid by him before the Commissioner of Public Works, and will also be considered by the City Council and the Mayor.

THE NEW WATER-WORKS RESERVOIR AT TRENTON, NEW JERSEY.

By Chas. A. Hague, M. Am. Soc. C. E.*

A new water-works reservoir, having some interesting features, was completed by the city of Trenton, N. J., in 1899. Mr. Lewis Lawton, of Trenton, N. J., was the contractor for the work, and the writer was designing and constructing engineer.

The reservoir has a capacity of 110,000,000 gallons. It is situated upon rather valuable land near the northwest edge of the city. The water on the flat portion of the bottom has a depth of 45 ft., and only 13 ft. of this is below the field level. The remaining 32 ft. is enclosed by a masonry wall and earth embankments (Figs. 1 and 2), built above the original level; that is to say, 29% of the basin is in excavation and 71% is in embankment.

Besides an increased storage capacity, it was desired to raise the flow level to the city as much as possible. After due consideration an increased head of 25 ft. over and above the old, or No. 1, basin was decided upon, this being all that was easily obtainable within the cost decided upon. Trenton has grown upon and is rapidly spreading over a slightly elevated plateau, and as there is no great elevation to the surrounding country thereabout, the problem became one of deciding how high into the air safety and the means at command would permit the work to be carried. There was another possible site about three miles further away from town, but as there was only 7 ft. additional head to be gained, and about all of this would be needed to overcome the friction of the water flowing the extra distance, to say nothing of $100,000 or more needed for additional water mains, the meager apparent gain in elevation was abandoned and the site near the city decided upon.

The field selected contains about 14 acres in one convenient, nearly level parcel of land. The first work was the sinking of several wells about 4 ft.

*Consulting Engineer, 39 Cortlandt St., New York city.

in diameter, so that the character and strata of the subsoil could be clearly determined. The material was found to consist of a very superior quality of hardpan, overlaying a hard close sandstone; it is a dense stratum of ocherous clay, solidly compacted with gravel and some sand. In some places the clay predominates, although a goodly proportion of gravel was nearly always present; in other portions the sand would seem to be the greater part of the mixture, but even in such sections the argillaceous binding element was sufficiently in evidence to ensure good material. A very considerable portion of the stratum, with slight tempering, would make a nearly ideal puddle; in fact, some portions were carefully selected and worked moist into a perfect puddle for ramming into the trench at the base of the walls.

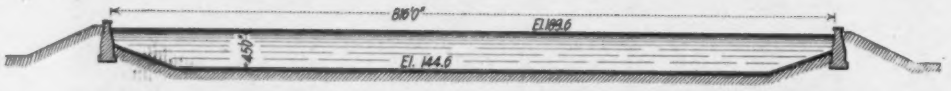


FIG. 1. LONGITUDINAL SECTION OF NEW RESERVOIR AT TRENTON, N. J.

The inner embankment, extending from the field level up to the masonry wall, or within 21 ft. of the high water line, is practically composed of puddle. The stripped surface of the ground was well rolled with a heavy grooved roller, and then the embankment built thereon, thoroughly rolled as it went up. After the inner embankment and the inner slopes in the solid earth were formed, as well as the flat bottom of the basin, the whole surface of these was thoroughly rolled, thus forming a solid mass of hardpan mixture, compacted, and practically impervious. Over the bottom and inner slopes there was placed 12 ins. of concrete, made of American Portland cement, crushed trap rock, and clean washed Delaware River sand. This concrete was laid in two thicknesses of 6 ins. each; the first layer, next the earth, was made with 1½-in. stone; and the top or finishing layer with fine stone siftings. The inner slope is 3 to 1 and the slope of the outside embankment is 2 to 1. The latter was built against the outside of the wall, and was partly composed of the same materials as the inner portions, although not so critically selected. The entire surface of the outer embankment was covered with 6 ins. of top soil.

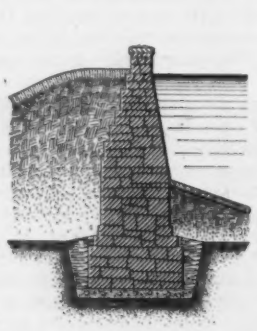


Fig. 2. Typical Cross-Section Through Masonry Wall.

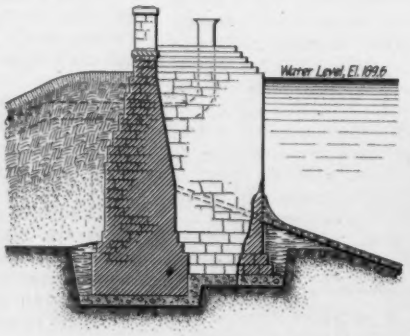


Fig. 3. Inlet Pipe at East End of Reservoir.

stone was to be had within economical distances, and preliminary sketches and calculations showed that the amount of excavation down to the bottom flow line would make a full embankment outside the wall, and an embankment within 21 ft. of the high water line inside the wall, the toe of the outer bank coming within 10 ft. of the boundaries of the property. So a combination was made with a stone wall of a section giving a fairly low center of gravity, safe from overthrow against the pressure regardless of any outer bank; then a full outer bank for insurance, and a low water bank thoroughly protecting the base of the wall. The desired increase of 25 ft. in the water head having been obtained within walls secure against overthrow, all that remained was to provide a tight bottom for the basin, which, being composed of gravelly clay, practically puddle, thoroughly rolled and covered with a foot of strong dense concrete, was not difficult of accomplishment.

The water is brought to the reservoir, as shown in Fig. 3, through a 36-in. force main, which terminates in a vertical bell-mouthed pipe, from whence it falls down a series of steps. By this means the water is aerated to a very considerable extent, and enters the basin with little disturbance of the water already in storage, thereby ensuring as far as possible the absence of special currents, and magnifying the chances of sedimentation before being taken out at the opposite end. The water leaves the basin about 700 ft. from where it enters, and, during atmospheric conditions most likely to produce anchor ice, at the protected end of the reservoir. There are two 48-in. outlet pipes, as shown by Figs. 4, 5 and 6, supported on stone piers, and surrounded by concrete and puddle. These pipes, one at an extreme low drainage level, and the other 25 ft. above the bottom, are united in the gate-house and provided with valves so that either or both may be used in connection with either or both of the street service mains shown. The water is usually taken from the upper pipe, thus accentuating the benefits of sedimentation as the water passes from one end of the reservoir to the other. The advantage of 25 ft. depth of water below the working outlet is thus combined with facilities for "blowing out" the sediment at the lower pipe whenever desired.

At present, at least, this reservoir is fairly large in proportion to the system it supplies, and one of the practical benefits of this fact is the chance for sedimentation afforded, and also cessation of pumping when the river is turbid. These features, combined with the points already noted in connection with the inlet and outlets, result in a very high quality of effluent as to clearness, as was at once noticed all over the city after the new reservoir had been placed in commission. A more specific statement relating to the improvement due to sedimentation is with the gage for turbidity showing 0.12 at the pumping station, the effluent from the basin at the end opposite the inlet has shown only 0.02.

The water was turned into the reservoir Sept. 20, 1899, and in the course of 30 days it was run up to within 10 ft. of the high level. Then it became apparent that seepage through the new concrete was taking place, although not to any serious extent, and on account of some sand streaks in the hardpan bottom, water slowly found its way to the outside, showing on the surface of the ground at different places. However, as the water level was easily maintained and the pump-house records showed no material increase, the matter was not considered more than a natural and temporary leakage for new concrete and masonry. When the excavation for the basin was commenced and down for about 10 ft., the "puddle" material continued all over the field, but below the 10-ft. spots, pockets, and streaks of sand commenced to develop, and finally at the complete depth, there were three or four places along the Pennington Ave. side which looked as though they might give trouble under 45 ft. head of water. Also, when the two 30-in. service mains were laid along the avenue a little below the level of the bottom of the reservoir, sand pockets corresponding to those within the basin about 200 ft. away were revealed. But as this sand stratum was very dense and hard, it was not considered at all dangerous, but rather presented a proposition as to whether it justified the necessary expenditure for shutting it off. Finally, for the sake of the chance of saving several thousand dollars, it was decided to go ahead to completion and then judge upon the matter of remedy after the fine silt which the water carried had an opportunity of asserting itself on the surface of the concrete.

The matter of asphalt lining was also considered, and some interesting experiments were tried with a slab of asphalt concrete. The slab was made of the thickness and consistency proposed to be used, and was 2 ft. long by 1 ft. broad. It was placed under water in a tank and supported at each end so it could be bent in the middle about 4 ins. below level; it was left so several weeks and then turned upside down and bent the other way about an equal length of time. Then it was immersed and kept in water in which cakes of ice were constantly floating, so as to represent possible conditions in winter, the bending and rebending being continued. It acted in a very satisfactory and promising manner as a reservoir lining, especially in a case like this one, where the asphalt concrete would never be exposed to the high temperatures of the summer months; and it showed that any settlement or loss of alinement in a reservoir bottom could be met by the adjustability, perfectly automatic, at any probable water temperatures, without crack or rupture.

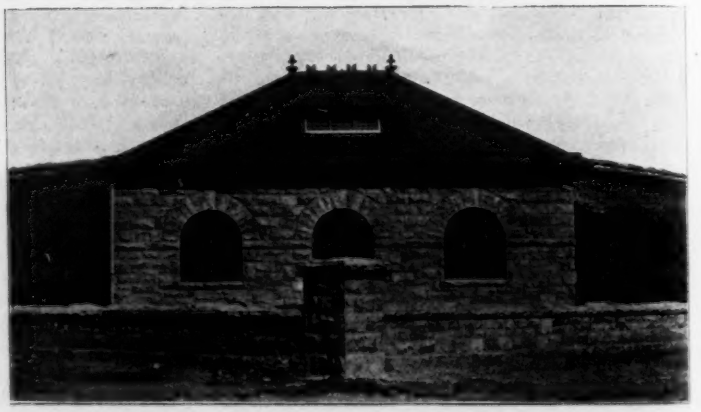


FIG. 7. VIEW OF GATE HOUSE AT TRENTON, N. J.

After water had been held in the basin at a depth of about 30 ft. for six weeks, it was emptied and examined; but everything was found to be sound and in good order, so it was again filled and put into service in December, 1899, continuing to supply the city to the present time. A blind drain of crushed stone was laid all around the reservoir, at the toe of the outer embankment,

This drain was 3 ft. deep and covered with a foot of earth, so the ground surface presented a normal appearance. The drain was included in the original design, but was omitted to save cost, although the sequal indicated it to be desirable. This encircling drain was tapped by two tile drains at opposite ends of the reservoir, and the seepage, together with the surface drainage, was carried away from the neighborhood of the basin about 1,500 ft. in each direction, emptying into a running brook. The surroundings of the reservoir immediately began to dry up, and in fact the fields thereabout, in the spring and fall, were drained as they never were before.

At first the drains showed an outflow of a volume which half filled the 6-in. outlet at one place; and with the water running about 2 ins. deep in a

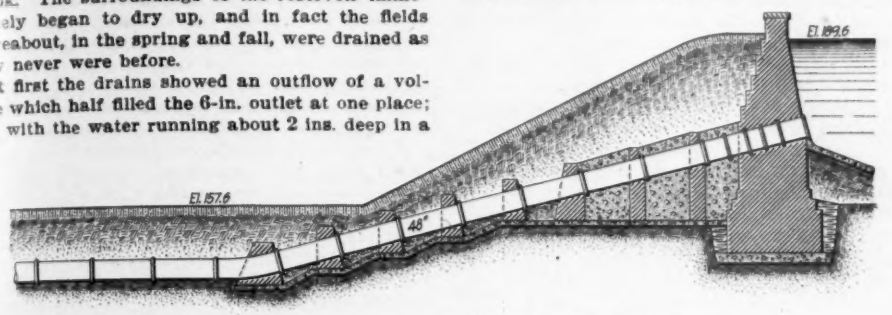


Fig.6. Upper or Service Outlet Pipe at West End of Reservoir.

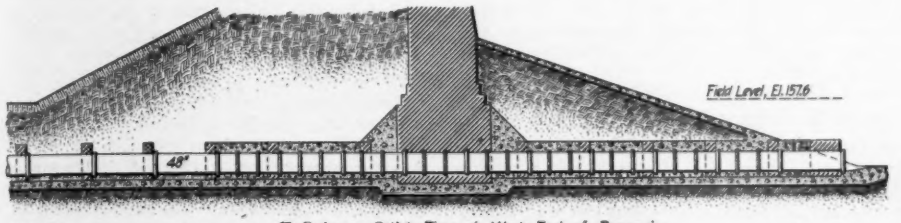


Fig.5. Lower Outlet Pipe at West End of Reservoir.

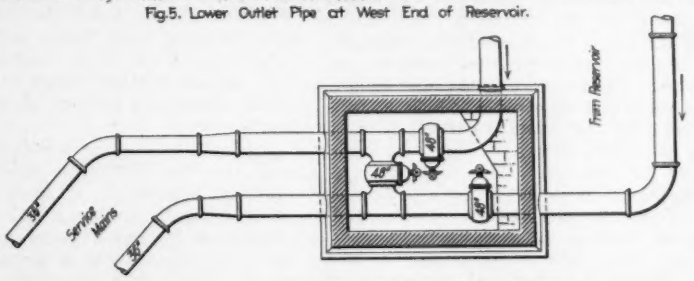


Fig.4. Sectional Plan of Gate House.

FIGS. 4, 5, 6. PLAN OF OUTLET GATE HOUSE AND DEVELOPED SECTIONS, SHOWING OUTLET PIPES.

10-in. drain at the other; so on considering that 110,000,000 gallons of water was in storage under a head of 45 ft., the seepage could not be said to be excessive. Part of the escaping water was, of course, the underdrainage of two extensive rather flat fields. In the course of seven or eight months the flow very nearly ceased; in fact, at times, entirely so, at the 6-in. drain, and probably dropped down to an amount representing mostly land drainage. As no other flowing water is in evidence, and the pumpage and effluent seem to be normal in quantity, there is evidently no further appreciable leakage from the basin.

When the sand streaks were first discovered, and their potency for mischief recognized, the idea of additional lining of course evolved itself, but when the basin was emptied after six weeks from the first water, the wet, impalpable silt spread like a velvet carpet over the entire bottom, gave promise of remedy which was finally realized; the drawing of the effluent so far above the bottom as 25 ft. no doubt insuring such a quietness of the body of water as to very materially affect the seepage for the better.

THE BRITISH PACIFIC CABLE is to land its Canadian end at Kelp Bay, near the entrance to Barclay Sound, and 100 miles from Victoria. Work has commenced in England on this cable, which is to be 5,834½ miles long. It is the longest ever made, and is to be transported in and laid from one ship, especially built for the purpose. The first stretch, from Victoria to Fanning Island, is 3,337 miles. The whole cable is to be laid by Jan. 1, 1903, and it will cost $10,000,000.

THE REMOVAL OF THE BOSTON GARBAGE reduction works from their present location near the Calf Pasture sewage pumping station at Dorchester to either Long Island or Rainsford Island has been recommended by a committee consisting of Wm. Jackson, M. Am. Soc. C. E., City Engineer, Bertrand T. Wheeler, Superintendent of Streets, and City Solicitor Babson. The committee proposes that the city donate the new site, sign a contract for ten years on the terms of the present contract, and pay the New England Sanitary Product Co. $140,000 for moving. The company would deed its present buildings and wharf to the city and construct new brick buildings; or, if it moved to Rainsford Island, it might move the old buildings, paying the city an appraised valuation of the same. The city now pays $47,000 a year for towing the garbage to the plant and for its disposal there.

THE FAMOUS ELECTROLYSIS SUIT AT PEORIA, Ill., has at last progressed to the point of a report by Mr. Frank L. Wean, Special Master in Chancery, in which he recommends that an injunction be issued in favor of the Peoria Water Co. The suit was brought over three years ago, against the street railway companies of Peoria, to prevent the further operation of electric cars in a manner injurious to the water mains. On May 26, 1898, Mr. Wean was appointed special master to take testimony. This he has done in much detail, in a number of cities. Judge Grosscup, of the U. S. Circuit Court, must next pass upon the findings of the master.

WATER PURIFICATION AT NEW HAVEN, CONN., is now under consideration. The New Haven Water Co. has engaged Mr. Geo. W. Fuller, Assoc. M. Am. Soc. C. E., to examine and report on the subject. Filtration has been discussed from time to time in the past few years, but the present action is said to be due to the recent epidemic of typhoid fever in one section of New Haven.

TWO BIDS FOR THE UPPER BELMONT FILTER plant at Philadelphia were received on May 28. For the filter plant alone Ryan & Kelley bid $999,000, and Daniel J. McNichol $1,099,166. For the reservoir, filters, clear water basin and various accessories the two contractors named bid $1,930,905 and $2,247,684, respectively. The lowest bidder requires 22 months and the highest only 10 months to complete the work. Five bids were submitted for a filtered water conduit from Torresdale, on the Delaware River, to the Frankford pumping station. These bids ranged from $1,251,206 to $3,350,000. The details of the bids for both of the pieces of work are given in our Construction News Supplement.

EXPORTS FROM U. S. PORTS SOUTH OF NEW YORK are discussed in the last bulletin of the Bureau of Statistics for the fiscal year 1900. During that year New Orleans has taken the second rank as an exporting city, and has displaced Boston. For the year mentioned the New Orleans exports amounted to $115,658,764; and for the nine months of 1901 the value amounted to $122,234,669; the Boston figures for these last nine months were $81,173,-051. In 1900 the port of New York handled 47.03% of the foreign commerce of the country, as compared with 52.5% in 1890; 51.4% in 1896, and 48% in 1897. This loss is principally in exportations, the imports remaining about the same as in 1894. Philadelphia's percentage of the total foreign commerce has remained almost stationary for four years, being 5.8% in 1900. Baltimore has gained, and its percentage was 8.28% of the total in 1900 as against 7.6% in 1895.

THE MASSACHUSETTS TOPOGRAPHICAL SURVEY Commission no longer exists; as Gov. Crane signed, on June 7, the bill abolishing this commission, referred to in our issue of June 6, and it is now law. In naming this commission and the Board of Harbor and Land Commissioners, we were in error in making the chief engineers members of the commission. Each commission had three members, who appointed a chief engineer.

COMPOSITE TIMBER COLUMNS.

By Halbert Powers Gillette.*

Among bridge engineers there still exists a diversity of opinion as to the safe load that may be imposed upon timber columns, which is not to be wondered at considering the comparatively few breakin tests thus far made on full-sized sticks. More particularly is the foregoing true relative to timber columns made of two or more sticks bolted together, as in the top chord of a wooden truss.

Composite timber columns will long continue to be built, even where timber of enormous size is available, as in Oregon and Washington, for the very good reason that large sticks rot at the heart, while the smaller sticks in composite columns dry out and last longer. The subject of composite columns is one, therefore, that should receive further theoretical and experimental investigation.

An examination of the strain sheets of Howe trusses built in years past seems to show that the designers used Smith's formula (Trautwine, p. 458), with a factor of safety of six, assuming the strength of a built-up column to be equal to that of a single solid stick of the same size as the combined sticks bolted together.

On the other hand, in Johnson's "Modern Framed Structures," p. 353, we read, in connection with the use of the modern straight line formula, based upon Prof. Lanza's experiments on full-sized sticks, that:

> In applying the (straight line) formula to composite columns made up of several sticks bolted together at intervals, give to each stick its proportionate share of the total load to be carried over that member, and then assume that it stands alone and unsupported. This is the only safe rule. Even though they are firmly bolted with packing blocks or washers notched into the sides, these grow loose in time and do not resist initial lateral bending. They should never be assumed to act as one solid stick.

With Smith's formula, a safety factor of six is common; with the modern straight line formula the safety factor is four; and the resultant dimensions of a composite column are practically the same whether designed by Smith's formula, assuming bolts to render the column as solid as a single stick, or by the modern straight line formula, assuming each stick to act separately.

The question that arises is whether or not the engineer using the modern straight line formula is justified in assuming that composite columns may be considered as acting like a single solid stick, as did the users of Smith's formula. The following deduction seems to indicate that this may be done under certain conditions. Where the ratio of the length of a stick to the dimension of its least side is small, failure takes place by direct crushing, and not by lateral deflection. In such cases bolting several sticks together cannot add to their strength. Where failure takes place by lateral deflection, however, bolting undoubtedly does add to the strength; and the amount of this lateral deflection under varying conditions must be ascertained before the engineer can intelligently trust to bolts to add to the strength of a built-up column.

We find in "Modern Framed Structures," p. 144, the formula for lateral deflection of a column with pin-end supports:

$$\Delta = \frac{1}{10} \frac{(f - p)\, l^2}{E\, y_1}, \text{ nearly.} \qquad (1)$$

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*791 Powers Block, Rochester, N. Y.

While on p. 147, as it is stated, we can put for l, $\frac{l}{2}$ where the ends are flat we have:

$$\Delta = \frac{1}{40} \frac{(f - p)\, l^2}{E\, y_1}. \qquad (2)$$

Δ = lateral deflection in inches at rupture.
f = unit stress in extreme fibers at rupture.
p = unit load on column $= \frac{P}{A}$.

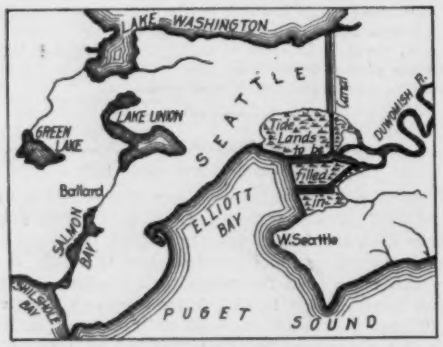


Fig. 1. Map Showing Method of Reclaiming Tidal Land in Elliott Bay, Seattle, Washington.

l = length of column in inches.
d = least side of rectangular column in inches.
$y_1 = \frac{d}{2}$ = least distance from neutral axis to extreme fiber.
E = modulus of elasticity = 1,250,000 for white pine.

On p. 151, we find for white pine with flat ends the straight line column formula, where

$$\frac{l}{d} \leq 60, \quad p = f - 0.6 \left(\frac{l}{d}\right)^2. \qquad (3)$$

which, substituted in eq. (2), gives

$$\Delta = \frac{0.03\, l^4}{E\, d^2} = \frac{l}{42{,}000{,}000} \left(\frac{l}{d}\right)^3 \qquad (4)$$

for white pine.

The following table is based on eq. (4):

When l = 10 ft. = 120 ins.

$\frac{l}{d}$ =	10	15	20	25	30	35	40	45	50
Δ =	.003	.01	.024	.044	.077	.12	.18	.26	.36

To obtain the deflection Δ for any other value of l simply multiply the tabular value of Δ by one-tenth of the length of the column. Thus, for a column whose length is 20 ft. the deflection where the ratio $\frac{l}{d} = 40$ is $0.18 \times \frac{20}{10} = 0.36$.

It is apparent that whether bolting is to be considered as making two sticks act together as one solid stick depends not only upon $\frac{l}{d}$, but upon l. When l is 10 ft. or less it is evident that in ordinary bridge structures bolts can be counted upon to furnish little or no assistance in resisting crushing: for even when $\frac{l}{d} = 40$ the deflection .18, or $\frac{1}{6}$-in., is so slight that the shrinkage of timber in drying might equal it, and so leave bolts comparatively useless. But when $l = 20$ and $\frac{l}{d} = 50$, we have $\Delta = .72$, or nearly $\frac{3}{4}$ in., an amount so large that bolts can be confidently counted upon to make two sticks (5 × 12 ins. × 20 ft.) bolted side by side, act as one solid stick. When $l = 30$ and $\frac{l}{d} = 45$, $\Delta = .78$ ins., so that two sticks 8 × 16 ins. × 30 ft. bolted side by side may be considered as acting like one stick 16 x 16 ins.

The greater the length of the column, the less may be the ratio $\frac{l}{d}$ at which bolts may be considered as rendering the built-up column practically solid.

For Howe truss chords we have reason to believe that our deduction is well on the side of safety, for it will be remembered that eq. (4) contains the condition that the column have flat ends. Now, according to Stoney, in "Strains and Stresses," p. 448, the members between panel points of a continuous upper chord act as if they were on knife-edge supports, in which case the deflection is four times that given in eq. (2), so that we should have used eq. (1) in our deduction. We cannot, however, multiply the values of Δ given in our table by four, because the straight line formula (3) was derived from experiments on flat-end columns. Until the strength of full-sized pin-ended wooded columns shall have been determined by experiment we shall have no means of investigating the matter further with much satisfaction. But our conclusions regarding the dimensions of sticks in built-up columns when bolts can be relied upon are on the side of safety in any case; and as we have seen, no fixed rule can be laid down as to where or when not to rely upon the bolts in a composite timber column. In each case the problem is to be solved along the lines indicated above.

FILLING IN TIDE-WATER FLATS AT SEATTLE, WASH.

The City of Seattle, Washington, to which so much attention has been attracted of late years by reason of its close connection with the Alaska gold discoveries, and the increased Oriental trade, is built upon the hills lying between Puget Sound and Lake Washington, there being very little flat ground for manufacturing purposes. Between the city proper and the suburb of West Seattle is a large area, a portion of Elliott Bay, which is overflowed at high tide, but which is entirely bare at extreme low tide; the amount of tide, from mean low water to extreme high tide, being 16.7 ft. These tide-flats are being filled with material taken from two waterways, as shown on the plan, Fig. 1, each one 1,000 ft. wide and 30 ft. in depth at low water. These waterways connect to the south with Duwamish River, and with a third waterway running east from the southern end of the east waterway to the foot of the hill, where it connects with the projected ship canal to Lake Washington, which is to be 140 ft. in width and 30 ft. deep. The difference in level between low water in the lake and high tide in the bay is to be overcome by a lock at the western end of the highland section, as shown in profile.

This work is being executed in accordance with plans furnished by Major Thos. W. Symons, U. S. Engineer Corps, which plans were approved by Col. Mendell and Gen. Craighill, of the U. S. Engineer Corps, and also by the late Col. Henry Flad, the distinguished engineer who served for a long time as a member of the United States Mississippi River Commission, and who was sent to Seattle by the Mississippi Valley Trust Co., of St. Louis, to make a thorough examination in its interests, pending negotiations for funds to begin the work. The plans have also been formally approved, and the work authorized to proceed by both the State and Federal governments.

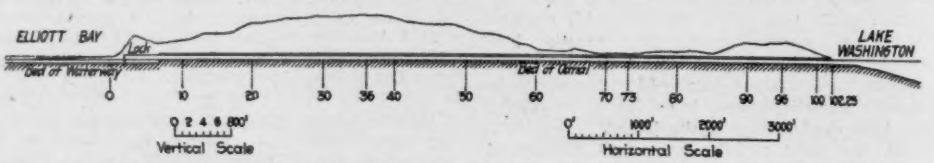


FIG. 2. PROFILE OF THE HIGHLAND SECTION OF THE SEATTLE AND LAKE WASHINGTON WATERWAY.

This work was mentioned in the Engineering News of March 15, and July 12, 1894; January 24 1895, and December 31, 1896, being then carried on by the Seattle & Lake Washington Waterway Co. Soon after the last-mentioned date, the Waterway Co. suspended operations owing to litigation as to the legality of the project, and a great deal of time was consumed in this legal controversy, but finally the highest courts of the State decided in favor of the Waterway Co. at every point.

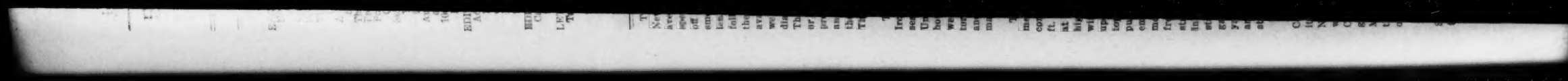
During the ensuing dull times no further progress was made in filling, but in the early part of the year 1900 arrangements were made to continue the work, and a contract was let to the Puget Sound Bridge & Dredging Co., of Seattle, Washington, to continue the work, and very satisfactory progress has been made during the last ten months.

The general plan of operation is to construct bulkhead lines along the margins of the waterways and fill in limited areas behind the same by hydraulic dredges, thus creating impregnable restraining works, and inclosing vast interior settling basins which will be filled with material from the highland section. Further contracts have been made, so that when the construction of the bulkheads with their backing of highlands has progressed to a sufficient extent, work will be begun on the highland section by the installment of powerful machinery for loosening the material by water under high pressure, and transporting it, by the returning water, in flumes and pipes, to the embankment on the tide flats. Material from the highest parts of the hill will be carried to the remotest districts, and the districts near by will be filled from the lower levels so as to make use of gravitation for carriage, in each case. Every yard of material that must be moved has a profitable place of deposit.

The average cost of filling these tide lands with solid materials, under the scheme of the Seattle & Lake Washington Waterway Co., is much less than the average cost of the perishable structures of wood and piles, now used to sustain frame houses over the water. The land created in this way will furnish hundreds of acres of perfectly level building sites, which will be traversed by railways, and abut upon deep water, thus affording a manufacturing district which will not be excelled by any city of the United States, unless it is Chicago.

A remarkable feature of this enterprise, to quote from the first opinion of Captain (now Major) Thos. W. Symons, U. S. Engineer Corps, is that "these gigantic and beneficial works will be accomplished without any tax on existing properties." The explanation of the above quotation is, that the values that are to sustain the charge, are created by the operation that changes the bottom of the bay, which, as such, is worthless, into dry land, and adds, out of nothing, to the taxable values of the city, in a ratio of three to one of the cost of the improvement. From the official report of the President of the Waterway Co. to the Commissioner of Public Lands for 1900, the following facts appear:

"The suit mentioned in the last report as pending on appeal, which was the fourth one that had been taken to the Supreme Court, was decided, as all the others had been, in favor of the contentions of the Waterway Company."

The financial scheme of the Waterway Co. was then reorganized by Hon. Will H. Parry, who had been for six consecutive years, Comptroller of Seattle, and who is now the manager of the Moran Bros. Co. ship-building plant. Accordingly, on Feb. 21, 1900, the Waterway Co. signed a contract with Morris & Whitehead, Bankers, of Philadelphia, Denver and Portland, for the taking up of its outstanding bonded indebtedness, the cancellation of all mortgages on its property and franchises, and the sale of tide land certificates, to be earned to the extent of $3,000,000, which will be sufficient to enable the entire project to be executed.

We are indebted to Mr. Eugene Semple, President of the Seattle & Lake Washington Waterway Co., for the above information.